Wang, L., Gong, C. "Abutments and Retaining Structures."
Bridge Engineering Handbook.
Ed. Wai-Fah Chen and Lian Duan
Boca Raton: CRC Press, 2000

29

Abutments and Retaining Structures

Linan Wang
California Transportation Department

Chao Gong
ICF Kaiser Engineers, Inc.

29.1 Introduction
29.2 Abutments
Abutment Types • General Design Considerations • Seismic Design Considerations • Miscellaneous Design Considerations • Design Example
29.3 Retaining Structures
Retaining Structure Types • Design Criteria • Cantilever Retaining Wall Design Example • Tieback Wall • Reinforced Earth-Retaining Structure • Seismic Consideration for Retaining Structures

29.1 Introduction

As a component of a bridge, the abutment provides the vertical support to the bridge superstructure at the bridge ends, connects the bridge with the approach roadway, and retains the roadway base materials from the bridge spans. Although there are numerous types of abutments and the abutments for the important bridges may be extremely complicated, the analysis principles and design methods are very similar. In this chapter the topics related to the design of conventional highway bridge abutments are discussed and a design example is illustrated.

Unlike the bridge abutment, the earth-retaining structures are mainly designed for sustaining lateral earth pressures. Those structures have been widely used in highway construction. In this chapter several types of retaining structures are presented and a design example is also given.

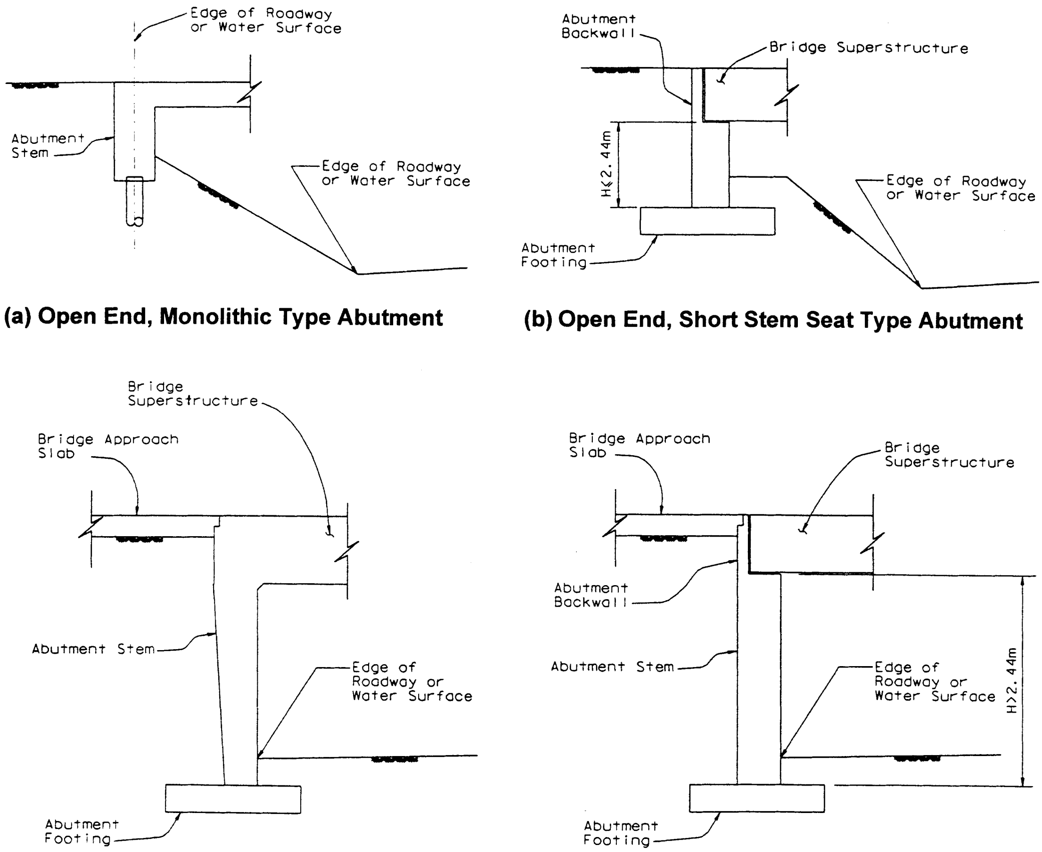
29.2 Abutments

29.2.1 Abutment Types

Open-End and Closed-End Abutments

From the view of the relation between the bridge abutment and roadway or water flow that the bridge overcrosses, bridge abutments can be divided into two categories: open-end abutment, and closed-end abutment, as shown in Figure 29.1.

For the open-end abutment, there are slopes between the bridge abutment face and the edge of the roadway or river canal that the bridge overcrosses. Those slopes provide a wide open area for the traffic flows or water flows under the bridge. It imposes much less impact on the environment



(a) Open End, Monolithic Type Abutment

(b) Open End, Short Stem Seat Type Abutment

(c) Close End, Monolithic Type Abutment

(d) Close End, High Stem Seat Type Abutment

FIGURE 29.1 Typical abutment types.

and the traffic flows under the bridge than a closed-end abutment. Also, future widening of the roadway or water flow canal under the bridge by adjusting the slope ratios is easier. However, the existence of slopes usually requires longer bridge spans and some extra earthwork. This may result in an increase in the bridge construction cost.

The closed-end abutment is usually constructed close to the edge of the roadways or water canals. Because of the vertical clearance requirements and the restrictions of construction right of way, there are no slopes allowed to be constructed between the bridge abutment face and the edge of roadways or water canals, and high abutment walls must be constructed. Since there is no room or only a little room between the abutment and the edge of traffic or water flow, it is very difficult to do the future widening to the roadways and water flow under the bridge. Also, the high abutment walls and larger backfill volume often result in higher abutment construction costs and more settlement of road approaches than for the open-end abutment.

Generally, the open-end abutments are more economical, adaptable, and attractive than the closed-end abutments. However, bridges with closed-end abutments have been widely constructed in urban areas and for rail transportation systems because of the right-of-way restriction and the large scale of the live load for trains, which usually results in shorter bridge spans.

Monolithic and Seat-Type Abutments

Based on the connections between the abutment stem and the bridge superstructure, the abutments also can be grouped in two categories: the monolithic or end diaphragm abutment and the seat-type abutment, as shown in Figure 29.1.

The monolithic abutment is monolithically constructed with the bridge superstructure. There is no relative displacement allowed between the bridge superstructure and abutment. All the superstructure forces at the bridge ends are transferred to the abutment stem and then to the abutment backfill soil and footings. The advantages of this type of abutment are its initial lower construction cost and its immediate engagement of backfill soil that absorbs the energy when the bridge is subjected to transitional movement. However, the passive soil pressure induced by the backfill soil could result in a difficult-to-design abutment stem, and higher maintenance cost might be expected. In the practice this type of abutment is mainly constructed for short bridges.

The seat-type abutment is constructed separately from the bridge superstructure. The bridge superstructure seats on the abutment stem through bearing pads, rock bearings, or other devices. This type of abutment allows the bridge designer to control the superstructure forces that are to be transferred to the abutment stem and backfill soil. By adjusting the devices between the bridge superstructure and abutment the bridge displacement can be controlled. This type of abutment may have a short stem or high stem, as shown in Figure 29.1. For a short-stem abutment, the abutment stiffness usually is much larger than the connection devices between the superstructure and the abutment. Therefore, those devices can be treated as boundary conditions in the bridge analysis. Comparatively, the high stem abutment may be subject to significant displacement under relatively less force. The stiffness of the high stem abutment and the response of the surrounding soil may have to be considered in the bridge analysis. The availability of the displacement of connection devices, the allowance of the superstructure shrinkage, and concrete shortening make this type of abutment widely selected for the long bridge constructions, especially for prestressed concrete bridges and steel bridges. However, bridge design practice shows that the relative weak connection devices between the superstructure and the abutment usually require the adjacent columns to be specially designed. Although the seat-type abutment has relatively higher initial construction cost than the monolithic abutment, its maintenance cost is relatively lower.

Abutment Type Selection

The selection of an abutment type needs to consider all available information and bridge design requirements. Those may include bridge geometry, roadway and riverbank requirements, geotechnical and right-of-way restrictions, aesthetic requirements, economic considerations, etc. Knowledge of the advantages and disadvantages for the different types of abutments will greatly benefit the bridge designer in choosing the right type of abutment for the bridge structure from the beginning stage of the bridge design.

29.2.2 General Design Considerations

Abutment design loads usually include vertical and horizontal loads from the bridge superstructure, vertical and lateral soil pressures, abutment gravity load, and the live-load surcharge on the abutment backfill materials. An abutment should be designed so as to withstand damage from the Earth pressure, the gravity loads of the bridge superstructure and abutment, live load on the superstructure or the approach fill, wind loads, and the transitional loads transferred through the connections between the superstructure and the abutment. Any possible combinations of those forces, which produce the most severe condition of loading, should be investigated in abutment design. Meanwhile, for the integral abutment or monolithic type of abutment the effects of bridge superstructure deformations, including bridge thermal movements, to the bridge approach structures must be

TABLE 29.1 Abutment Design Loads (Service Load Design)

Abutment Design Loads	Case I	II	III	IV	V
Dead load of superstructure	X	X	—	X	X
Dead load of wall and footing	X	X	X	X	X
Dead load of earth on heel of wall including surcharge	X	X	X	X	—
Dead load of earth on toe of wall	X	X	X	X	—
Earth pressure on rear of wall including surcharge	X	X	X	X	—
Live load on superstructure	X	—	—	X	—
Temperature and shrinkage	—	—	—	X	—
Allowable pile capacity of allowable soil pressure in % or basic	100	100	150	125	150

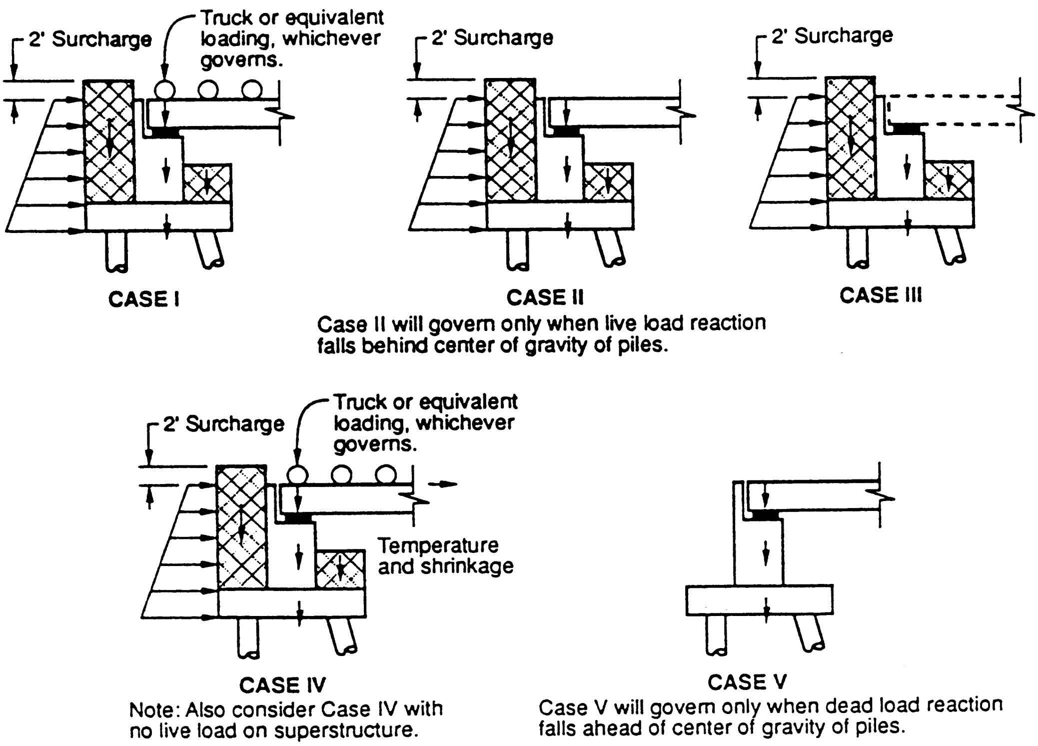


FIGURE 29.2 Configuration of abutment design load and load combinations.

considered in abutment design. Nonseismic design loads at service level and their combinations are shown in Table 29.1 and Figure 29.2. It is easy to obtain the factored abutment design loads and load combinations by multiplying the load factors to the loads at service levels. Under seismic loading, the abutment may be designed at no support loss to the bridge superstructure while the abutment may suffer some damages during a major earthquake.

The current AASHTO Bridge Design Specifications recommend that either the service load design or the load factor design method be used to perform an abutment design. However, due to the uncertainties in evaluating the soil response to static, cycling, dynamic, and seismic loading, the service load design method is usually used for abutment stability checks and the load factor method is used for the design of abutment components.

The load and load combinations listed in Table 29.1 may cause abutment sliding, overturning, and bearing failures. Those stability characteristics of abutment must be checked to satisfy certain restrictions. For the abutment with spread footings under service load, the factor of safety to resist sliding should be greater than 1.5; the factor of safety to resist overturning should be greater than 2.0; the factor of safety against soil bearing failure should be greater than 3.0. For the abutment with pile support, the piles have to be designed to resist the forces that cause abutment sliding, overturning, and bearing failure. The pile design may utilize either the service load design method or the load factor design method.

The abutment deep shear failure also needs to be studied in abutment design. Usually, the potential of this kind of failure is pointed out in the geotechnical report to the bridge designers. Deep pilings or relocating the abutment may be used to avoid this kind of failure.

29.2.3 Seismic Design Considerations

Investigations of past earthquake damage to the bridges reveal that there are commonly two types of abutment earthquake damage — stability damage and component damage.

Abutment stability damage during an earthquake is mainly caused by foundation failure due to excessive ground deformation or the loss of bearing capacities of the foundation soil. Those foundation failures result in the abutment suffering tilting, sliding, settling, and overturning. The foundation soil failure usually occurs because of poor soil conditions, such as soft soil, and the existence of a high water table. In order to avoid these kinds of soil failures during an earthquake, borrowing backfill soil, pile foundations, a high degree of soil compaction, pervious materials, and drainage systems may be considered in the design.

Abutment component damage is generally caused by excessive soil pressure, which is mobilized by the large relative displacement between the abutment and its backfilled soil. Those excessive pressures may cause severe damage to abutment components such as abutment back walls and abutment wingwalls. However, the abutment component damages do not usually cause the bridge superstructure to lose support at the abutment and they are repairable. This may allow the bridge designer to utilize the deformation of abutment backfill soil under seismic forces to dissipate the seismic energy to avoid the bridge losing support at columns under a major earthquake strike.

The behavior of abutment backfill soil deformed under seismic load is very efficient at dissipating the seismic energy, especially for the bridges with total length of less than 300 ft (91.5 m) with no hinge, no skew, or that are only slightly skewed (i.e., <15°). The tests and analysis revealed that if the abutments are capable of mobilizing the backfill soil and are well tied into the backfill soil, a damping ratio in the range of 10 to 15% is justified. This will elongate the bridge period and may reduce the ductility demand on the bridge columns. For short bridges, a damping reduction factor, *D*, may be applied to the forces and displacement obtained from bridge elastic analysis which generally have damped ARS curves at 5% levels. This factor *D* is given in Eq. (29.1).

$$D = \frac{1.5}{40\,C + 1} + 0.5 \tag{29.1}$$

where C = damping ratio.

Based on Eq. (29.1), for 10% damping, a factor $D = 0.8$ may be applied to the elastic force and displacement. For 15% damping, a factor $D = 0.7$ may be applied. Generally, the reduction factor D should be applied to the forces corresponding to the bridge shake mode that shows the abutment being excited.

The responses of abutment backfill soil to the seismic load are very difficult to predict. The study and tests revealed that the soil forces, which are applied to bridge abutment under seismic load, mainly depend on the abutment movement direction and magnitude. In the design practice, the Mononobe–Okabe method usually is used to quantify those loads for the abutment with no restraints on the top. Recently, the "near full scale" abutment tests performed at the University of California at Davis show a nonlinear relationship between the abutment displacement and the backfill soil reactions under certain seismic loading when the abutment moves toward its backfill soil. This relation was plotted as shown in Figure 29.3. It is difficult to simulate this nonlinear relationship between the abutment displacement and the backfill soil reactions while performing bridge dynamic analysis. However, the tests concluded an upper limit for the backfill soil reaction on the abutment. In design practice, a peak soil pressure acting on the abutment may be predicted corresponding to certain abutment displacements. Based on the tests and investigations of past earthquake damages, the California Transportation Department suggests guidelines for bridge analysis considering abutment damping behavior as follows.

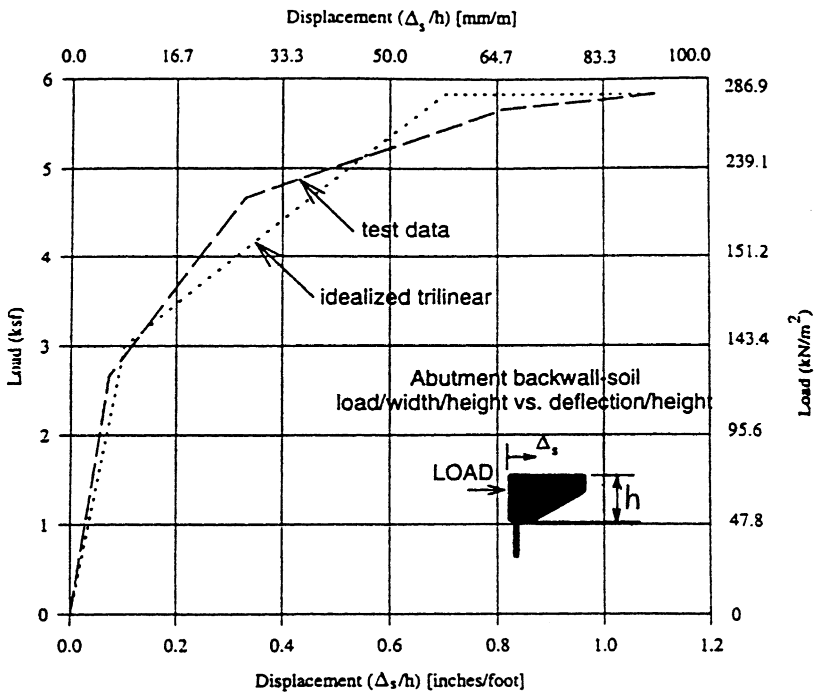


FIGURE 29.3 Proposed characteristics and experimental envelope for abutment backfill load–deformation.

By using the peak abutment force and the effective area of the mobilized soil wedge, the peak soil pressure is compared to a maximum capacity of 7.7 ksf (0.3687 MPa). If the peak soil pressure exceeds the soil capacity, the analysis should be repeated with reduced abutment stiffness. It is important to note that the 7.7 ksf (0.3687 MPa) soil pressure is based on a reliable minimum wall height of 8 ft (2.438 m). If the wall height is less than 8 ft (2.438 m), or if the wall is expected to shear off at a depth below the roadway less than 8 ft (2.438 m), the allowable passive soil pressure must be reduced by multiplying 7.7 ksf (0.3687 MPa) times the ratio of (L/8) [2], where L is the effective height of the abutment wall in feet. Furthermore, the shear capacity of the abutment wall diaphragm (the structural member mobilizing the soil wedge) should be compared with the demand shear forces to ensure the soil mobilizations. Abutment spring displacement is then evaluated against an acceptable level of displacement of 0.2 ft (61 mm). For a monolithic-type abutment this displacement is equal to the bridge superstructure displacement. For seat-type abutments this displacement usually does not equal the bridge superstructure displacement, which may include the gap between the bridge superstructure and abutment backwall. However, a net displacement of about 0.2 ft (61 mm) at the abutment should not be exceeded. Field investigations after the 1971 San Fernando earthquake revealed that the abutment, which moved up to 0.2 ft (61 mm) in the longitudinal direction into the backfill soil, appeared to survive with little need for repair. The abutments in which the backwall breaks off before other abutment damage may also be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated.

For seismic loads in the transverse direction, the same general principles still apply. The 0.2-ft (61-mm) displacement limit also applies in the transverse direction, if the abutment stiffness is expected to be maintained. Usually, wingwalls are tied to the abutment to stiffen the bridge transversely. The lateral resistance of the wingwall depends on the soil mass that may be mobilized by the wingwall. For a wingwall with the soil sloped away from the exterior face, little lateral resistance can be predicted. In order to increase the transverse resistance of the abutment, interior supplemental shear walls may be attached to the abutment or the wingwall thickness may be increased, as shown in Figure 29.4. In some situations larger deflection may be satisfactory if a reasonable load path can be provided to adjacent bents and no collapse potential is indicated. [2]

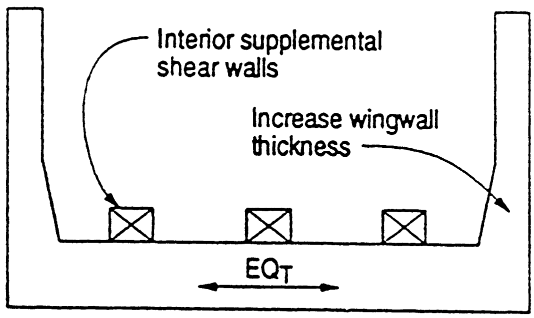
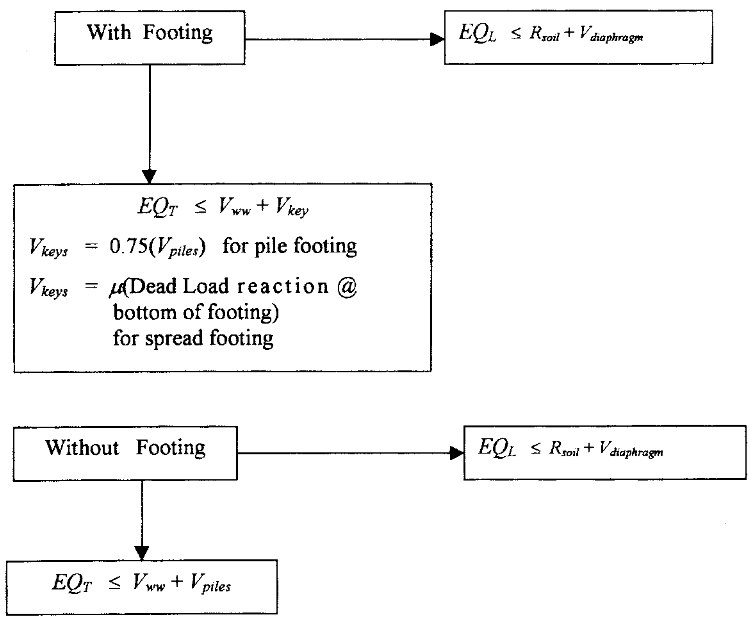


FIGURE 29.4 Abutment transverse enhancement.

Based on the above guidelines, abutment analysis can be carried out more realistically by a trial-and-error method on abutment soil springs. The criterion for abutment seismic resistance design may be set as follows.

Monolithic Abutment or Diaphragm Abutment (Figure 29.5)



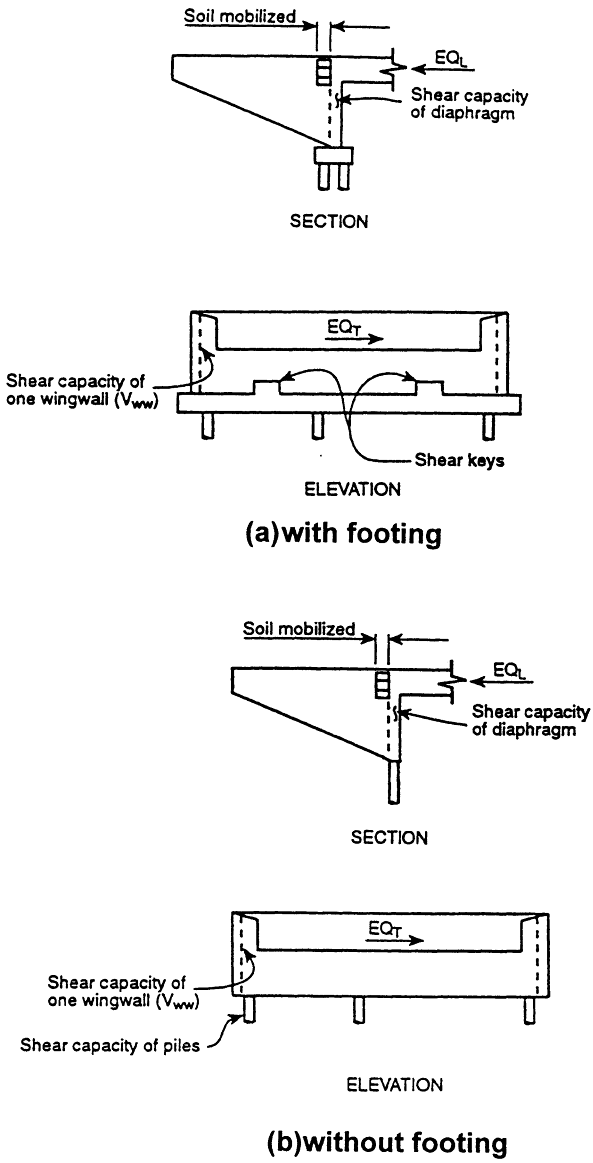
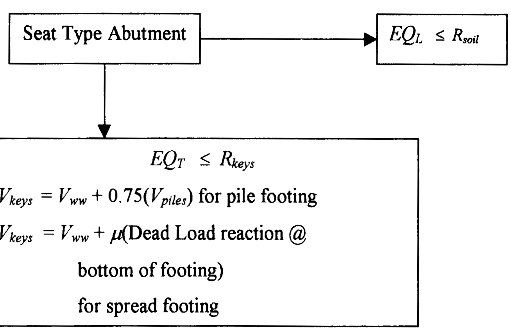


FIGURE 29.5 Seismic resistance elements for monolithic abutment.

Seat-Type Abutment (Figure 29.6)



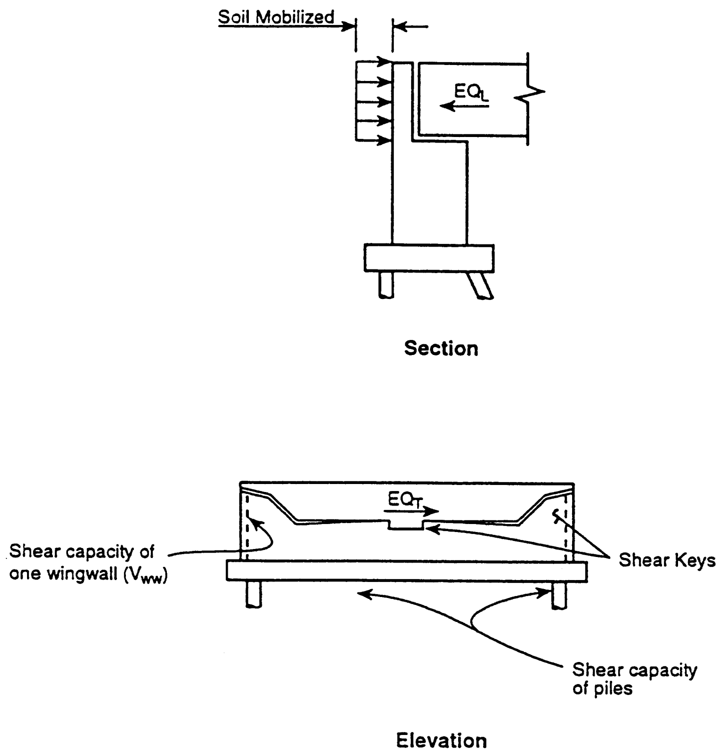


FIGURE 29.6 Seismic resistance elements for seat-type abutment.

where

EQ_L = longitudinal earthquake force from an elastic analysis
EQ_T = transverse earthquake force from an elastic analysis
R_{soil} = resistance of soil mobilized behind abutment
$R_{diaphragm}$ = φ times the nominal shear strength of the diaphragm
R_{ww} = φ times the nominal shear strength of the wingwall
R_{piles} = φ times the nominal shear strength of the piles
R_{keys} = φ times the nominal shear strength of the keys in the direction of consideration
φ = strength factor for seismic loading
μ = coefficient factor between soil and concrete face at abutment bottom

It is noted that the purpose of applying a factor of 0.75 to the design of shear keys is to reduce the possible damage to the abutment piles. For all transverse cases, if the design transverse earthquake force exceeds the sum of the capacities of the wingwalls and piles, the transverse stiffness for the analysis should equal zero ($EQ_T = 0$). Therefore, a released condition which usually results in larger lateral forces at adjacent bents should be studied.

Responding to seismic load, bridges usually accommodate a large displacement. To provide support at abutments for a bridge with large displacement, enough support width at the abutment must be designed. The minimum abutment support width, as shown in Figure 29.7, may be equal to the bridge displacement resulting from a seismic elastic analysis or be calculated as shown in Equation (29-2), whichever is larger:

$$N = (305 + 2.5L + 10H)(1 + 0.002\ S^2) \tag{29.2}$$

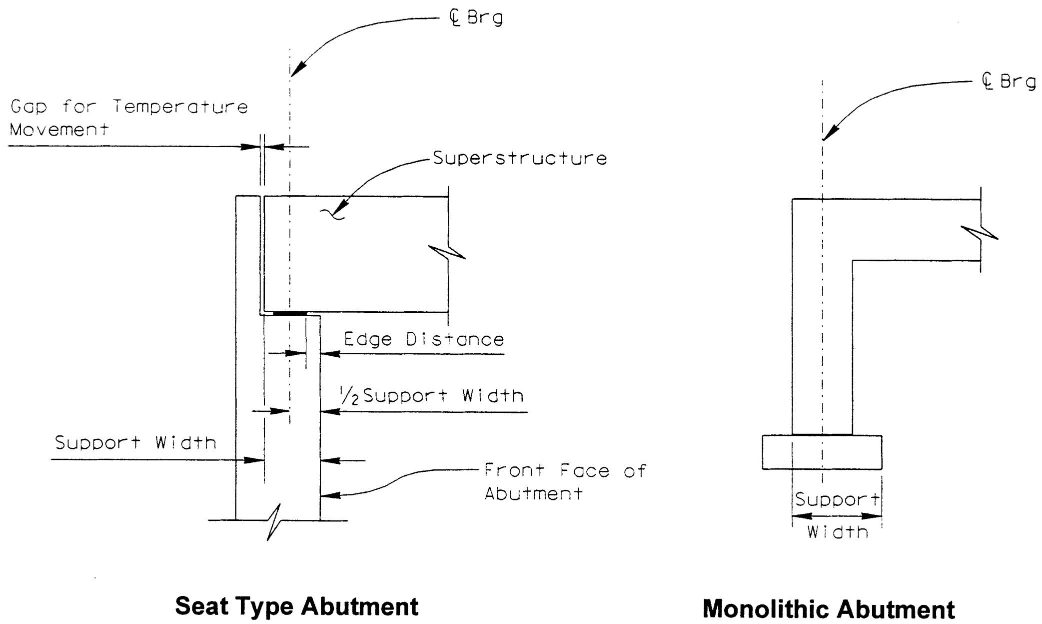


FIGURE 29.7 Abutment support width (seismic).

where

N = support width (mm)

L = length (m) of the bridge deck to the adjacent expansion joint, or to the end of bridge deck; for single-span bridges L equals the length of the bridge deck

S = angle of skew at abutment in degrees

H = average height (m) of columns or piers supporting the bridge deck from the abutment to the adjacent expansion joint, or to the end of the bridge deck; $H = 0$ for simple span bridges

29.2.4 Miscellaneous Design Considerations

Abutment Wingwall

Abutment wingwalls act as a retaining structure to prevent the abutment backfill soil and the roadway soil from sliding transversely. Several types of wingwall for highway bridges are shown in Figure 29.8. A wingwall design similar to the retaining wall design is presented in Section 29.3. However, live-load surcharge needs to be considered in wingwall design. Table 29.2 lists the live-load surcharge for different loading cases. Figure 29.9 shows the design loads for a conventional cantilever wingwall. For seismic design, the criteria in transverse direction discussed in Section 29.2.3 should be followed. Bridge wingwalls may be designed to sustain some damage in a major earthquake, as long as bridge collapse is not predicted.

Abutment Drainage

A drainage system is usually provided for the abutment construction. The drainage system embedded in the abutment backfill soil is designed to reduce the possible buildup of hydrostatic pressure, to control erosion of the roadway embankment, and to reduce the possibility of soil liquefaction during an earthquake. For a concrete-paved abutment slope, a drainage system also needs to be provided under the pavement. The drainage system may include pervious materials, PSP or PVC pipes, weep holes, etc. Figure 29.10 shows a typical drainage system for highway bridge construction.

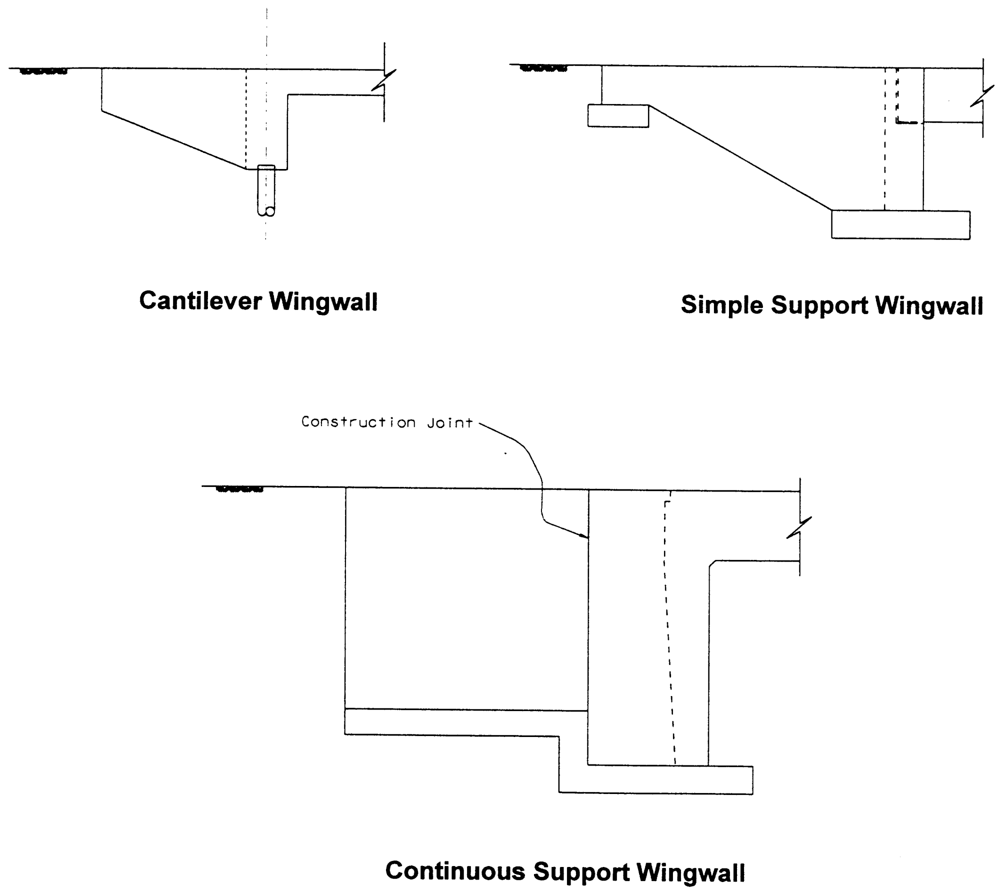


FIGURE 29.8 Typical wingwalls.

TABLE 29.2 Live Load Surcharges for Wingwall Design

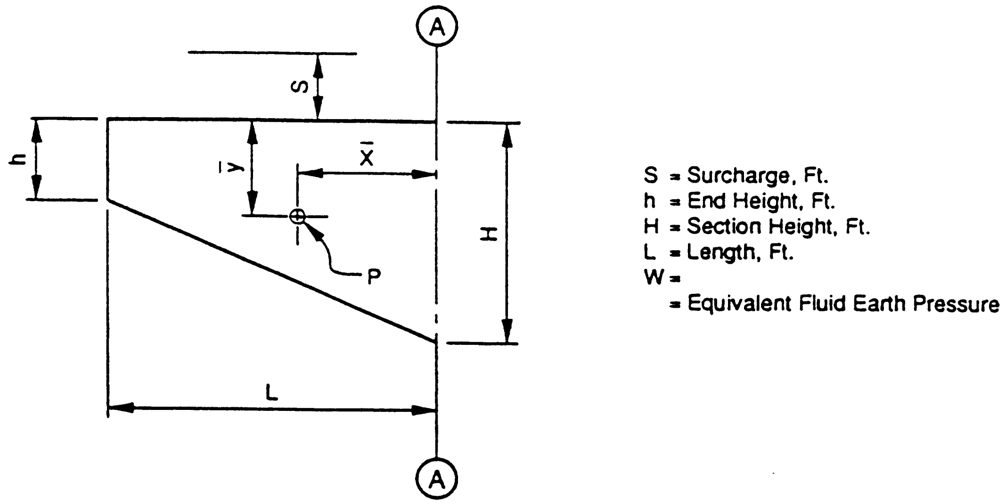
Highway truck loading	2 ft 0 in. (610 mm) equivalent soil
Rail loading E-60	7 ft 6 in. (2290 mm) equivalent soil
Rail loading E-70	8 ft 9 in. (2670 mm) equivalent soil
Rail loading E-80	10 ft 0 in. (3050 mm) equivalent soil

Abutment Slope Protection

Flow water scoring may severely damage bridge structures by washing out the bridge abutment support soil. To reduce water scoring damage to the bridge abutment, pile support, rock slope protection, concrete slope paving, and gunite cement slope paving may be used. Figure 29.11 shows the actual design of rock slope protection and concrete slope paving protection for bridge abutments. The stability of the rock and concrete slope protection should be considered in the design. An enlarged block is usually designed at the toe of the protections.

Miscellaneous Details

Some details related to abutment design are given in Figure 29.12. Although they are only for regular bridge construction situations, those details present valuable references for bridge designers.



$$M_{AA} = \frac{WL^2}{24}\left[3h^2 + (H + 4S)(H + 2h)\right]$$

$$P = \frac{WL}{6}\left[H^2 + (h + H)(h + 3S)\right]$$

$$\overline{X} = \frac{M_{AA}}{P}$$

FIGURE 29.9 Design loading for cantilever wingwall.

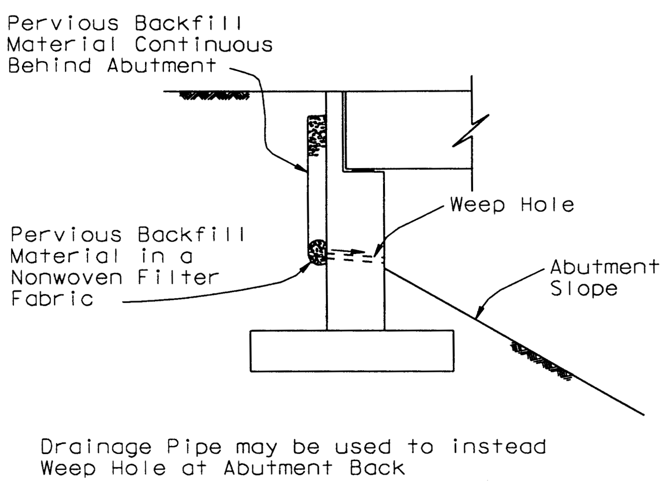
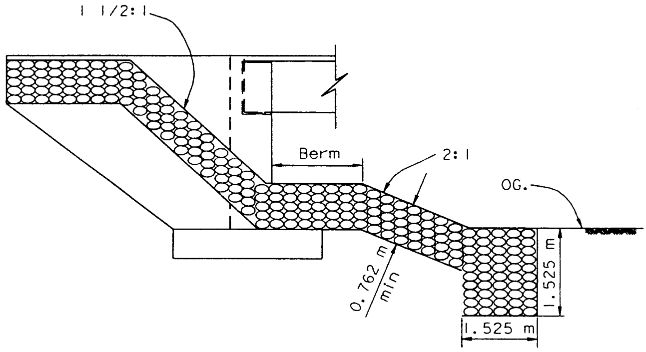


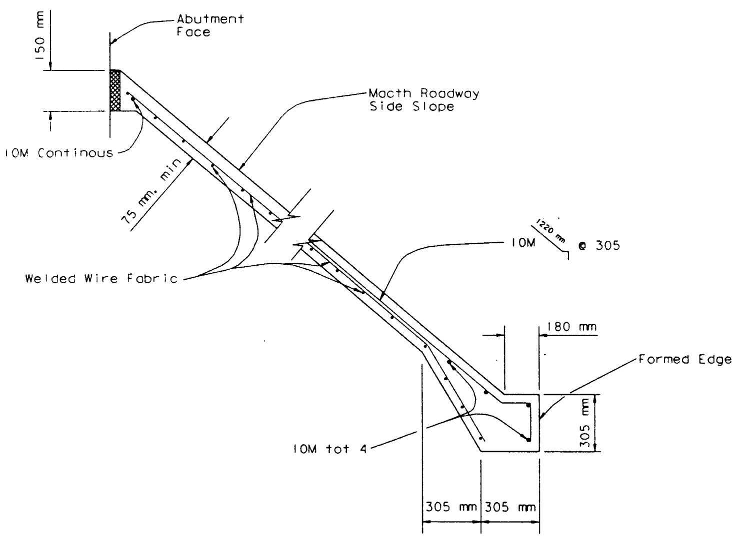
FIGURE 29.10 Typical abutment drainage system.

29.2.5 Design Example

A prestressed concrete box-girder bridge with 5° skew is proposed overcrossing a busy freeway as shown in Figure 29.13. Based on the roadway requirement, geotechnical information, and the details mentioned above, an open-end, seat-type abutment is selected. The abutment in transverse direction is 89 ft (27.13 m) wide. From the bridge analysis, the loads on abutment and bridge displacements are as listed bellow:



Rock Slope Protection



Concrete Slope Protection

FIGURE 29.11 Typical abutment slope protections.

Superstructure dead load	= 1630 kips (7251 kN)
HS20 live load	= 410 kips (1824 kN)
1.15 P-load + 1.0 HS load	= 280 kips (1245 kN)
Longitudinal live load	= 248 kips (1103 kN)
Longitudinal seismic load (bearing pad capacity)	= 326 kips (1450 kN)
Transverse seismic load	= 1241 kips (5520 kN)
Bridge temperature displacement	= 2.0 in. (75 mm)
Bridge seismic displacement	= 6.5 in. (165 mm)

Geotechnical Information

Live-load surcharge	= 2 ft (0.61 m)
Unit weight of backfill soil	= 120 pcf (1922 kg/m^3)

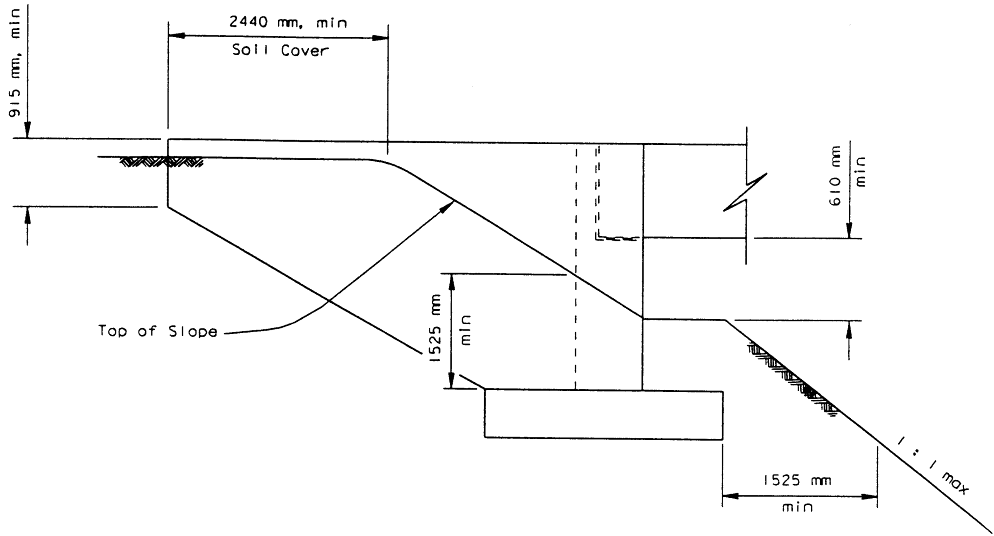


FIGURE 29.12 Abutment design miscellaneous details.

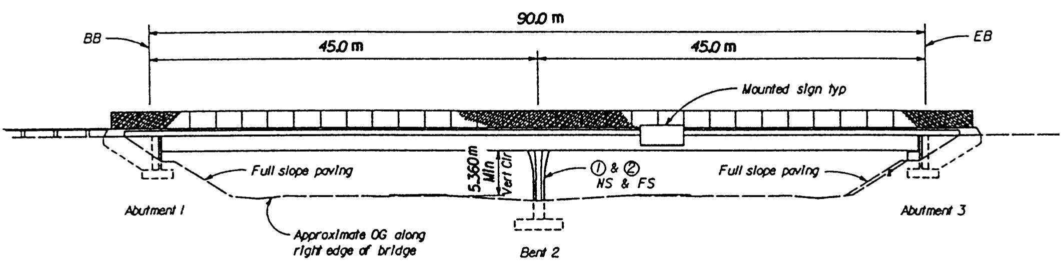


FIGURE 29.13 Bridge elevation (example).

Allowable soil bearing pressure	= 4.0 ksf (0.19 MPa)
Soil lateral pressure coefficient (Ka)	= 0.3
Friction coefficient	= tan 33°
Soil liquefaction potential	= very low
Ground acceleration	= 0.3 g

Design Criteria

Abutment design	Load factor method
Abutment stability	Service load method

Design Assumptions

1. Superstructure vertical loading acting on the center line of abutment footing;
2. The soil passive pressure by the soil at abutment toe is neglected;
3. 1.0 feet (0.305 m) wide of abutment is used in the design;
4. reinforcement yield stress, f_y = 60000 psi (414 MPa)
5. concrete strength, f_c' = 3250 psi (22.41 MPa)
6. abutment backwall allowed damage in the design earthquake

Solution

1. **Abutment Support Width Design**
   Applying Eq. (29.2) with

$$L = 6.5 \text{ m}$$

$$H = 90.0 \text{ m}$$

$$S = 5°$$

the support width will be $N = 600$ mm. Add 75 mm required temperature movement, the total required support width equals 675 mm. The required minimum support width for seismic case equals the sum of the bridge seismic displacement, the bridge temperature displacement, and the reserved edge displacement (usually 75 mm). In this example, this requirement equals 315 mm, not in control. Based on the 675-mm minimum requirement, the design uses 760 mm, OK. A preliminary abutment configuration is shown in Figure 29.14 based on the given information and calculated support width.

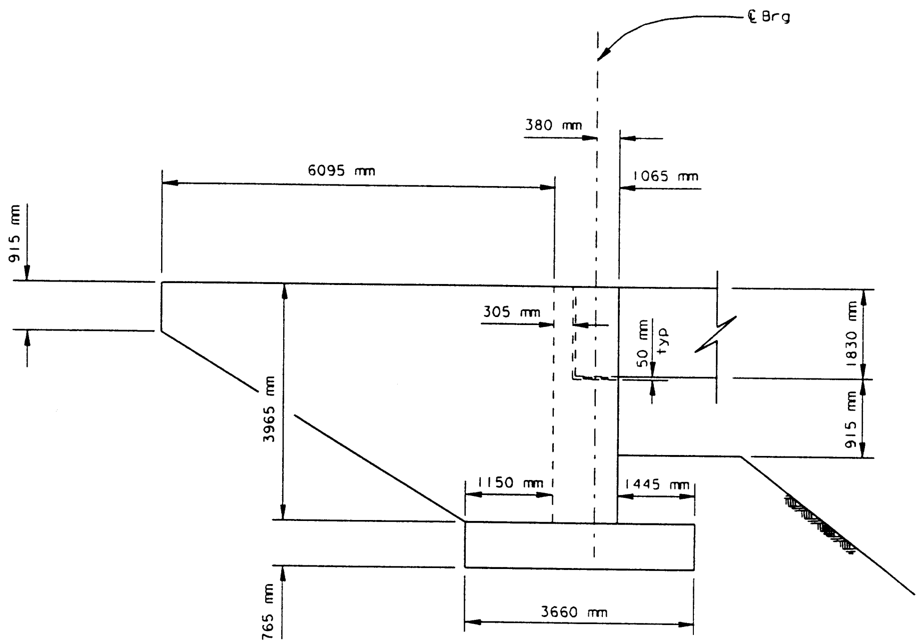


FIGURE 29.14 Abutment configuration (example).

2. **Abutment Stability Check**
   Figure 29.15 shows the abutment force diagram,

   where

$_{sc}q$ = soil lateral pressure by live-load surcharge
$_{e}q$ = soil lateral pressure
$_{eq}q$ = soil lateral pressure by seismic load
$_{DL}P$ = superstructure dead load
$_{HS}P$ = HS20 live load
$_{P}P$ = permit live load

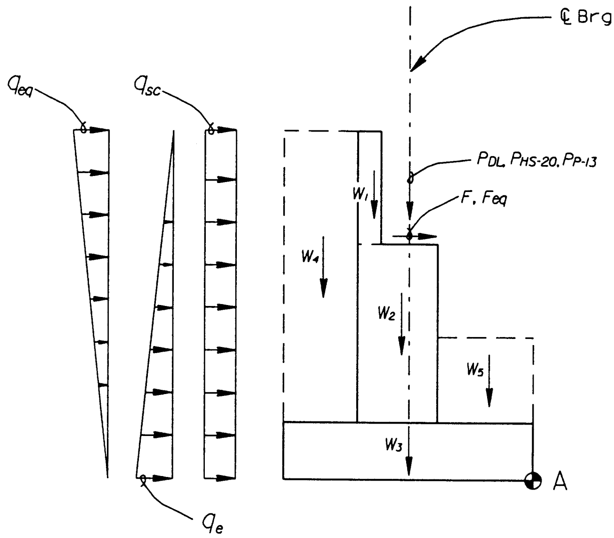


FIGURE 29.15 Abutment applying forces diagram (example).

F = longitudinal live load
F_{eq} = longitudinal bridge seismic load
P_{ac} = resultant of active seismic soil lateral pressure
h_{sc} = height of live-load surcharge
γ = unit weight of soil
W_i = weight of abutment component and soil block
$q_{sc} = k_a \times \gamma \times h_{sc} = 0.3 \times 0.12 \times 2 = 0.072$ ksf (0.0034 MPa)
$q_e = k_a \times \gamma \times H = 0.3 \times 0.12 \times 15.5 = 0.558$ ksf (0.0267 MPa)
$q_{eq} = k_{ae} \times \gamma \times H = 0.032 \times 0.12 \times 15.5 = 0.06$ ksf (0.003 MPa)

The calculated vertical loads, lateral loads, and moment about point A are listed in Table 29.3. The maximum and minimum soil pressure at abutment footing are calculated by

$$p = \frac{P}{B}\left(1 \pm \frac{6e}{B}\right) \tag{29.3}$$

where
p = soil bearing pressure
P = resultant of vertical forces
B = abutment footing width
e = eccentricity of resultant of forces and the center of footing

$$e = 2B - \frac{M}{P} \tag{29.4}$$

M = total moment to point A

Referring to the Table 29.1 and Eqs. (29.3) and (29.4) the maximum and minimum soil pressures under footing corresponding to different load cases are calculated as
Since the soil bearing pressures are less than the allowable soil bearing pressure, the soil bearing stability is OK.

Load Case	p_{max}	p_{min}	$p_{allowable}$ with Allowable % of Overstress	Evaluate
I	3.81	3.10	4.00	OK
II	3.42	2.72	4.00	OK
III	1.84	1.22	6.00	OK
IV	4.86	2.15	5.00	OK
V	2.79	1.93	6.00	OK
Seismic	6.73	0.54	8.00	OK

TABLE 29.3 Vertical Forces, Lateral Forces, and Moment about Point *A* (Example)

Load Description	Vertical Load (kips)	Lateral Load (kips)	Arm to *A* (ft)	Moment to *A* (k-ft)
Backwall W_1	0.94	—	7.75	7.28
Stem W_2	3.54	—	6.00	23.01
Footing W_3	4.50	—	6.00	27.00
Backfill soil	5.85	—	10.13	59.23
	—	4.33	5.17	–22.34
Soil surcharge	—	1.16	7.75	–8.65
Front soil W_4	1.71	—	2.38	4.06
Wingwalls	0.85	—	16.12	13.70
Keys	0.17	—	6.00	1.04
P_{DL}	18.31	—	6.00	110.00
P_{HS}	4.61	—	6.00	27.64
P_P	3.15	—	6.00	18.90
F	—	2.79	9.25	–25.80
F_{eq}	—	3.66	9.25	–33.90
Soil seismic load	—	0.47	9.30	–4.37

Check for the stability resisting the overturning (load case III and IV control):

Load Case	Driving Moment	Resist Moment	Factor of Safety	Evaluate
III	31	133.55	4.3	OK
IV	56.8	262.45	4.62	OK

Checking for the stability resisting the sliding (load case III and IV control)

Load Case	Driving Force	Resist Force	Factor of Safety	Evaluation
III	5.44	11.91	2.18	OK
IV	8.23	20.7	3.26	OK

Since the structure lateral dynamic force is only combined with dead load and static soil lateral pressures, and the factor of safety FS = 1.0 can be used, the seismic case is not in control.

3. **Abutment Backwall and Stem Design**

Referring to AASHTO guidelines for load combinations, the maximum factored loads for abutment backwall and stem are

Location	*V* (kips)	*M* (k-ft)
Backwall level	1.95	4.67
Bottom of stem	10.36	74.85

Abutment Backwall

Try #5 at 12 in. (305 mm) with 2 in. (50 mm) clearance

$d = 9.7$ in. (245 mm)

$$A_s \times f_y = 0.31 \times 60 \times \frac{12}{16} = 13.95 \text{ kips } (62.05 \text{ kN})$$

$$a = \frac{A_s \cdot f_y}{\phi \cdot f_c' \cdot b_w} = \frac{13.95}{(0.85)(3.25)(12)} = 0.42 \text{ in. } (10.67 \text{ mm})$$

$$M_u = \phi \cdot M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2}\right) = 0.9 \times 13.95 \times \left(9.7 - \frac{0.42}{2}\right) = 9.33 \text{ k} \cdot \text{ft } (13.46 \text{ kN} \cdot \text{m})$$

$$> 4.67 \text{ k} \cdot \text{ft } (6.33 \text{ kN} \cdot \text{m}) \quad \text{OK}$$

$$V_c = 2\sqrt{f_c'} \cdot b_w \cdot d = 2 \times \sqrt{3250} \times 12 \times 9.7 = 13.27 \text{ kips} \quad (59.03 \text{ kN})$$

$$V_u = \phi \cdot V_c = 0.85 \times 13.27 = 11.28 \text{ kip } (50.17 \text{ kN}) > 1.95 \text{ kips } (8.67 \text{ kN}) \quad \text{OK}$$

No shear reinforcement needed.

Abutment Stem

Abutment stem could be designed based on the applying moment variations along the abutment wall height. Here only the section at the bottom of stem is designed.

Try #6 at 12 in. (305 mm) with 2 in. (50 mm) clearance.

$$A_s \times f_y = 0.44 \times 60 = 26.40 \text{ kips } (117.43 \text{ kN})$$

$d = 39.4$ in. (1000 mm)

$$a = \frac{A_s \cdot f_y}{\phi \cdot f_c' \cdot b_w} = \frac{26.4}{(0.85)(3.25)(12)} = 0.796 \text{ in } (20.0 \text{ mm})$$

$$M_u = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2}\right) = 0.9 \times 26.4 \times \left(39.4 - \frac{0.8}{2}\right) = 77.22 \text{ k·ft} \quad (104.7 \text{kN·m})$$

$$> 74.85 \text{ k·ft} \quad (101.5 \text{kN·m}) \quad \text{OK}$$

$$V_c = 2\sqrt{f_c'}\, b_w\, d = 2 \times \sqrt{3250} \times 12 \times 39.4 = 53.91 \text{ kips} \quad (238 \text{ kN})$$

$$V_u = \phi \cdot V_c = 0.85 \times 53.91 = 45.81 \text{ kips} \quad (202.3 \text{ kN}) \quad > \quad 10.36 \text{ kips} \quad (46.08 \text{ kN}) \quad \text{OK}$$

No shear reinforcement needed.

4. Abutment Footing Design

Considering all load combinations and seismic loading cases, the soil bearing pressure diagram under the abutment footing are shown in Figure 29.16.

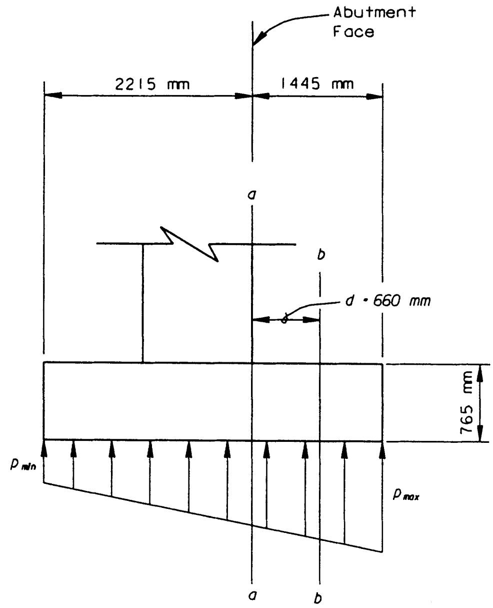


FIGURE 29.16 Bearing pressure under abutment footing (example).

a. *Design forces*:
   Section at front face of abutment stem (design for flexural reinforcement):

$$q_{a\text{-}a} = 5.1263 \text{ ksf } (0.2454 \text{ MPa})$$

$$M_{a\text{-}a} = 69.4 \text{ k-ft } (94.1 \text{ kN·m})$$

   Section at $d = 30 - 3 - 1 = 26$ in. (660 mm) from the front face of abutment stem (design for shear reinforcement):

$$q_{b\text{-}b} = 5.2341 \text{ ksf } (0.251 \text{ MPa})$$

$$V_{b\text{-}b} = 15.4 \text{ kips } (68.5 \text{ kN})$$

b. *Design flexural reinforcing* (*footing bottom*):
   Try #8 at 12, with 3 in. (75 mm) clearance at bottom

$$d = 30 - 3 - 1 = 26 \text{ in. } (660 \text{ mm})$$

$$A_s \times f_y = 0.79 \times 60 = 47.4 \text{ kips} \quad (211 \text{ kN})$$

$$a = \frac{A_s \cdot f_y}{\phi \cdot f_c' \cdot b_w} = \frac{47.4}{(0.85)(3.25)(12)} = 1.43 \text{ in.} \quad (36 \text{ mm})$$

$$M_n = \phi \cdot A_s \cdot f_y \left(d - \frac{a}{2} \right) = 0.9 \times 47.4 \times \left(26 - \frac{1.43}{2} \right) = 89.9 \text{ k·ft} \quad (121.89 \text{ kN·m})$$

$$> 69.4 \text{ k·ft} \quad (94.1 \text{ kN·m}) \qquad \text{OK}$$

$$V_c = 2\sqrt{f_c'} \cdot b_w \cdot d = 2 \times \sqrt{3250} \times 12 \times 26 = 35.57 \text{ kips} \quad (158.24 \text{ kN})$$

$$V_u = \phi \cdot V_c = 0.85 \times 35.57 = 30.23 \text{ kips} \quad (134.5 \text{ kN}) \quad > \quad 15.5 \text{ kips} \quad (68.5 \text{ kN}) \qquad \text{OK}$$

No shear reinforcement needed.

Since the minimum soil bearing pressure under the footing is in compression, the tension at the footing top is not the case. However, the minimum temperature reinforcing, 0.308 in.^{2}/ft (652 mm^2/m) needs to be provided. Using #5 at 12 in. (305 mm) at the footing top yields

$$A_s = 0.31 \text{ in.}^2/\text{ft}, \ (656 \text{ mm}^2/\text{m}) \qquad \text{OK}$$

5. Abutment Wingwall Design

The geometry of wingwall is

$$h = 3.0 \text{ ft } (915 \text{ mm}); \qquad S = 2.0 \text{ ft } (610 \text{ mm});$$

$$H = 13.0 \text{ ft } (3960 \text{ mm}); \qquad L = 18.25 \text{ ft } (5565 \text{ mm})$$

Referring to the Figure 29.15, the design loads are

$$V_{A-A} = \frac{wL}{6}\left[H^2 + (h+H)(h+3S)\right]$$

$$= \frac{0.36 \times 18.25}{6}\left[13^2 + (3+13)(3+3\times 2)\right] = 34 \text{ kips} \quad (152.39 \text{ kN})$$

$$M_{A-A} = \frac{wL^2}{24}\left[3h^2 + (H+4S)(H+2h)\right]$$

$$= \frac{0.036 \times 18.25^2}{24}\left[3(3)^2 + (13+4\times 2)(12+2\times 3)\right] = 212.8 \text{ k·ft} \quad (3129 \text{ kN·m})$$

Design flexural reinforcing. Try using # 8 at 9 (225 mm).

$$A_s \times f_y = 13 \times (0.79) \times 60 \times \frac{12}{9} = 821.6 \text{ kips} \quad (3682 \text{ kN})$$

$$a = \frac{A_s \cdot f_y}{\phi \cdot f_c' \cdot b_w} = \frac{1280}{(0.85)(3.25)(13)(12)} = 2.97 \text{ in.} \quad (75 \text{ mm})$$

$$d = 12 - 2 - 0.5 = 9.5 \text{ in. } (240 \text{ mm})$$

$$M_n = \phi \cdot A_s \cdot f_y\left(d - \frac{a}{2}\right) = 0.9 \times (821.6) \times \left(9.5 - \frac{2.97}{2}\right) = 493.8 \text{ k·ft } (7261 \text{ kN}\cdot\text{m})$$

$$> 212.8 \text{ k·ft} \quad (3129 \text{ kN}\cdot\text{m})$$

Checking for shear

$$V_c = 2\sqrt{f_c'} \cdot b_w \cdot d = 2 \times \sqrt{3250} \times 13 \times 12 \times 9.5 = 168 \text{ kips} \quad (757.3 \text{ kN})$$

$$V_u = \varphi \cdot V_c = 0.85 \times 168 = 142 \text{ kips} \quad (636 \text{ kN}) > 34 \text{ kips} \ (152.3 \text{kN}) \qquad \text{OK}$$

No shear reinforcing needed.

Since the wingwall is allowed to be broken off in a major earthquake, the adjacent bridge columns have to be designed to sustain the seismic loading with no wingwall resistance. The abutment section, footing, and wingwall reinforcing details are shown in Figures 29.17a and b.

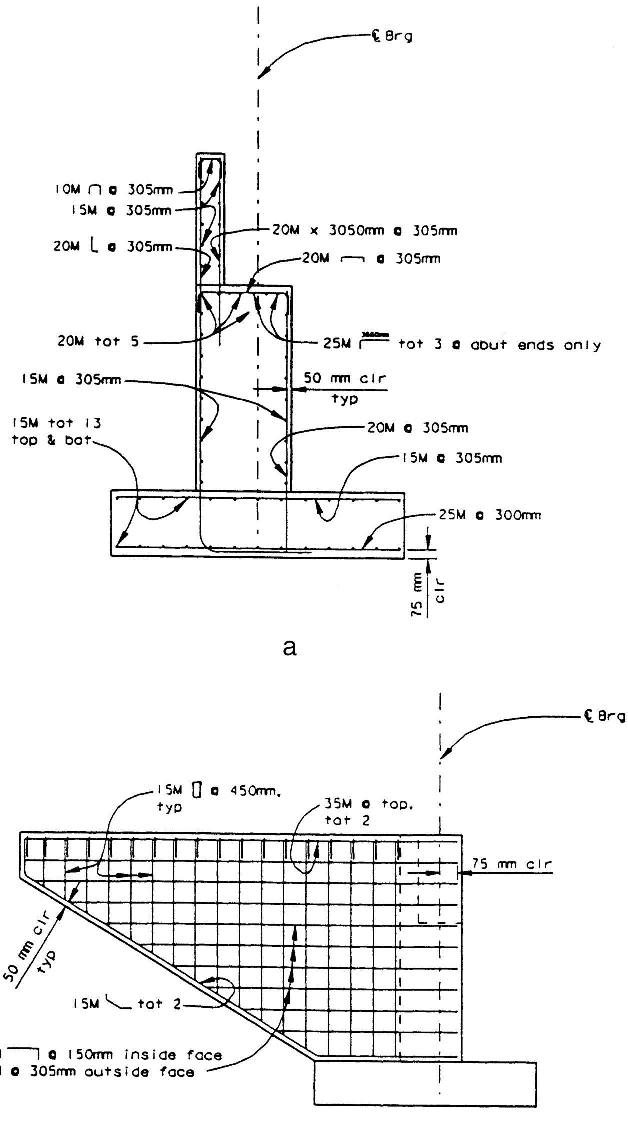


FIGURE 29.17 (a) Abutment typical section design (example). (b) Wingwall reinforcing (example).

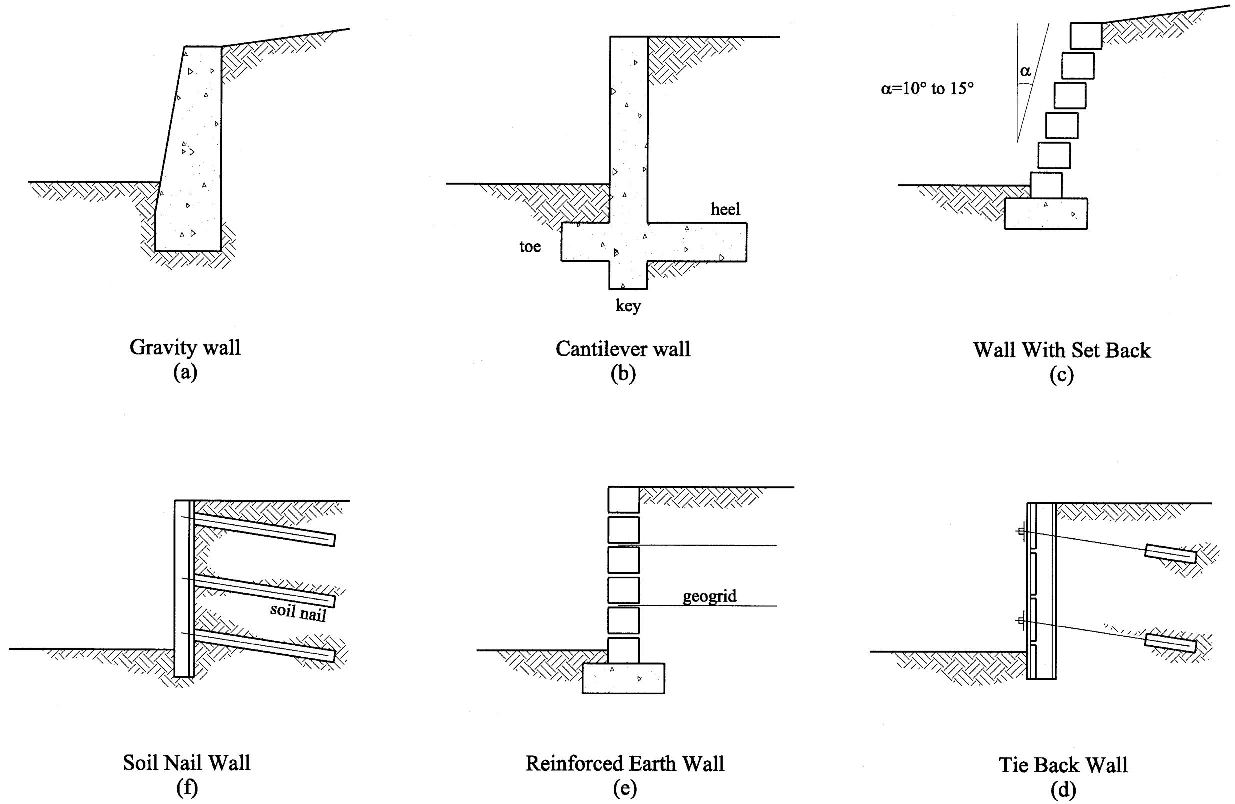


FIGURE 29.18 Retaining wall types.

29.3 Retaining Structures

29.3.1 Retaining Structure Types

The retaining structure, or, more specifically, the earth-retaining structure, is commonly required in a bridge design project. It is common practice that the bridge abutment itself is used as a retaining structure. The cantilever wall, tieback wall, soil nail wall and mechanically stabilized embankment (MSE) wall are the most frequently used retaining structure types. The major design function of a retaining structure is to resist lateral forces.

The cantilever retaining wall is a cantilever structure used to resist the active soil pressure in topography fill locations. Usually, the cantilever earth-retaining structure does not exceed 10 m in height. Some typical cantilever retaining wall sections are shown in Figure 29.18a.

The tieback wall can be used for topography cutting locations. High-strength tie strands are extended into the stable zone and act as anchors for the wall face elements. The tieback wall can be designed to have minimum lateral deflection. Figure 29.18d shows a tieback wall section.

The MSE wall is a kind of "reinforced earth-retaining" structure. By installing multiple layers of high-strength fibers inside of the fill section, the lateral deflection of filled soil will be restricted. There is no height limit for an MSE wall but the lateral deflection at the top of the wall needs to be considered. Figure 29.18e shows an example of an MSE wall.

The soil nail wall looks like a tieback wall but works like an MSE wall. It uses a series of soil nails built inside the soil body that resist the soil body lateral movement in the cut sections. Usually, the soil nails are constructed by pumping cement grout into predrilled holes. The nails bind the soil together and act as a gravity soil wall. A typical soil nail wall model is shown in Figure 29.18f.

29.3.2 Design Criteria

Minimum Requirements

All retaining structures must be safe from vertical settlement. They must have sufficient resistance against overturning and sliding. Retaining structures must also have adequate strength for all structural components.

1. *Bearing capacity*: Similar to any footing design, the bearing capacity factor of safety should be ≥1.0. Table 29.4 is a list of approximate bearing capacity values for some common materials. If a pile footing is used, the soil-bearing capacity between piles is not considered.
2. *Overturning resistance*: The overturning point of a typical retaining structure is located at the edge of the footing toe. The overturning factor of safety should be ≥1.50. If the retaining structure has a pile footing, the fixity of the footing will depend on the piles only.
3. *Sliding resistance*: The factor of safety for sliding should be ≥1.50. The typical retaining wall sliding capacity may include both the passive soil pressure at the toe face of the footing and the friction forces at the bottom of the footing. In most cases, friction factors of 0.3 and 0.4

TABLE 29.4 Bearing Capacity

	Bearing Capacity [N]	
Material	min, kPa	max, kPa
Alluvial soils	24	48
Clay	48	190
Sand, confined	48	190
Gravel	95	190
Cemented sand and gravel	240	480
Rock	240	—

can be used for clay and sand, respectively. If battered piles are used for sliding resistance, the friction force at the bottom of the footing should be neglected.

4. Structural strength: Structural section moment and shear capacities should be designed following common strength factors of safety design procedures.

Figure 29.19 shows typical loads for cantilever retaining structure design.

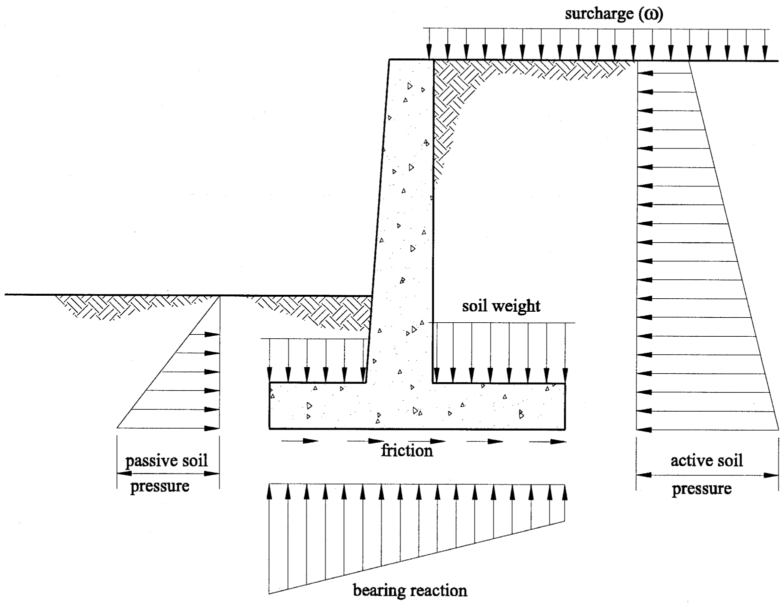


FIGURE 29 19 Typical loads on retaining wall.

Lateral Load

The unit weight of soil is typically in the range of 1.5 to 2.0 ton/m^3. For flat backfill cases, if the backfill material is dry, cohesionless sand, the lateral earth pressure (Figure 29.20a) distribution on the wall will be as follows

The active force per unit length of wall (Pa) at bottom of wall can be determined as

$$p_a = k_a \gamma H \tag{29.5}$$

The passive force per unit length of wall (Pa) at bottom of wall can be determined as

$$p_p = k_p \gamma H \tag{29.6}$$

where

H = the height of the wall (from top of the wall to bottom of the footing)
γ = unit weight of the backfill material
k_a = active earth pressure coefficient
k_p = passive earth pressure coefficient

The coefficients k_a and k_p should be determined by a geologist using laboratory test data from a proper soil sample. The general formula is

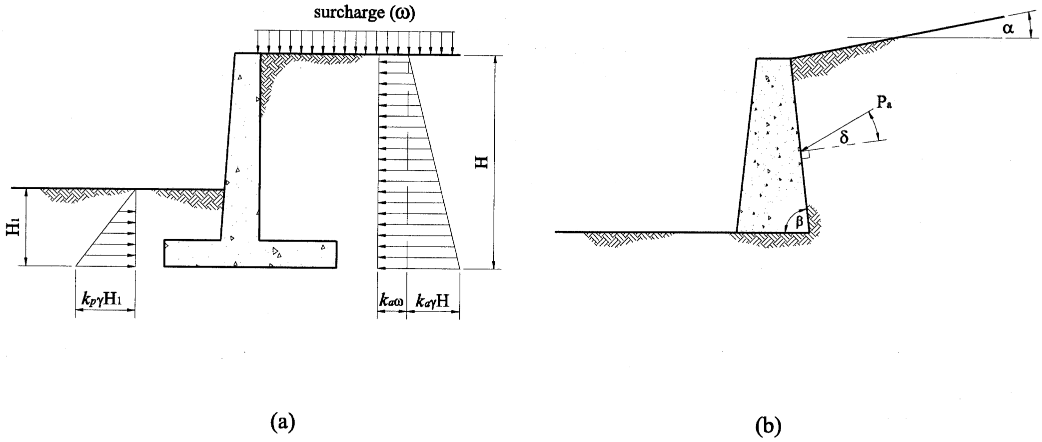


FIGURE 29.20 Lateral Earth pressure.

$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} \quad k_p = \frac{1}{k_a} = \frac{1 + \sin\phi}{1 - \sin\phi} \tag{29.7}$$

where ϕ is the internal friction angle of the soil sample.

Table 29.5 lists friction angles for some typical soil types which can be used if laboratory test data is not available. Generally, force coefficients of $k_a \geq 0.30$ and $k_p \leq 1.50$ should be used for preliminary design.

TABLE 29.5 Internal Friction Angle and Force Coefficients

Material	ϕ(*degrees*)	k_a	k_p
Earth, loam	30–45	0.33–0.17	3.00–5.83
Dry sand	25–35	0.41–0.27	2.46–3.69
Wet sand	30–45	0.33–0.17	3.00–5.83
Compact Earth	15–30	0.59–0.33	1.70–3.00
Gravel	35–40	0.27–0.22	3.69–4.60
Cinders	25–40	0.41–0.22	2.46–4.60
Coke	30–45	0.33–0.17	3.00–5.83
Coal	25–35	0.41–0.27	2.46–3.69

Based on the triangle distribution assumption, the total active lateral force per unit length of wall should be

$$P_a = \frac{1}{2} k_a \gamma H^2 \tag{29.8}$$

The resultant earth pressure always acts at distance of $H/3$ from the bottom of the wall.

When the top surface of backfill is sloped, the k_a coefficient can be determined by the Coulomb equation: (see Figure 29.20):

$$\mathrm{k_a} = \frac{\sin^2(\phi+\beta)}{\sin^2\beta \sin(\beta-\delta)\left[1 + \sqrt{\dfrac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\sin(\beta-\delta)\sin(\alpha+\beta)}}\right]^2} \tag{29.9}$$

Note that the above lateral earth pressure calculation formulas do not include water pressure on the wall. A drainage system behind the retaining structures is necessary; otherwise the proper water pressure must be considered.

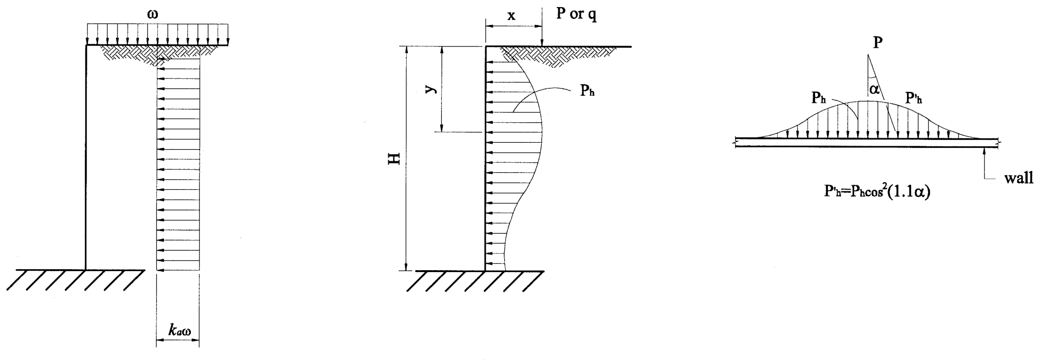
Table 29.6 gives values of k_a for the special case of zero wall friction.

TABLE 29.6 Active Stress Coefficient k_a Values from Coulomb Equation ($\delta = 0$)

		α					
		0.00°	18.43°	21.80°	26.57°	33.69°	45.00°
ϕ	β_o	Flat	1 to 3.0	1 to 2.5	1 to 2.0	1 to 1.5	1 to 1.0
20°	90°	0.490	0.731				
	85°	0.523	0.783				
	80°	0.559	0.842				
	75°	0.601	0.913				
	70°	0.648	0.996				
25°	90°	0.406	0.547	0.611			
	85°	0.440	0.597	0.667			
	80°	0.478	0.653	0.730			
	75°	0.521	0.718	0.804			
	70°	0.569	0.795	0.891			
30°	90°	0.333	0.427	0.460	0.536		
	85°	0.368	0.476	0.512	0.597		
	80°	0.407	0.530	0.571	0.666		
	75°	0.449	0.592	0.639	0.746		
	70°	0.498	0.664	0.718	0.841		
35°	90°	0.271	0.335	0.355	0.393	0.530	
	85°	0.306	0.381	0.404	0.448	0.602	
	80°	0.343	0.433	0.459	0.510	0.685	
	75°	0.386	0.492	0.522	0.581	0.781	
	70°	0.434	0.560	0.596	0.665	0.897	
40°	90°	0.217	0.261	0.273	0.296	0.352	
	85°	0.251	0.304	0.319	0.346	0.411	
	80°	0.287	0.353	0.370	0.402	0.479	
	75°	0.329	0.408	0.429	0.467	0.558	
	70°	0.375	0.472	0.498	0.543	0.651	
45°	90°	0.172	0.201	0.209	0.222	0.252	0.500
	85°	0.203	0.240	0.250	0.267	0.304	0.593
	80°	0.238	0.285	0.297	0.318	0.363	0.702
	75°	0.277	0.336	0.351	0.377	0.431	0.832
	70°	0.322	0.396	0.415	0.446	0.513	0.990

Any surface load near the retaining structure will generate additional lateral pressure on the wall. For highway-related design projects, the traffic load can be represented by an equivalent vertical surcharge pressure of 11.00 to 12.00 kPa. For point load and line load cases (Figure 29.21), the following formulas can be used to determine the additional pressure on the retaining wall:

For point load:



Uniform Surcharge
(a)

Point Load or Line Load
(b)

Horizontal Pressure Distribution
of Point Load
(c)

FIGURE 29.21 Additional lateral earth pressure. (a) Uniform surcharge; (b) point or line load; (c) horizontal pressure distribution of point load.

$$p_h = \frac{1.77V}{H^2}\frac{m^2n^2}{\left(m^2+n^2\right)^3} \quad (\mathrm{m} \le 0.4) \quad p_h \frac{0.28V}{H^2}\frac{m^2n^2}{\left(0.16+n^3\right)} \quad (m > 0.4) \qquad (29.10)$$

For line load:

$$p_h = \frac{\pi}{4}\frac{w}{H}\frac{m^2n}{\left(m^2+n^2\right)^2} \quad (m \le 0.4) \quad p_h = \frac{w}{H}\frac{0.203n}{\left(0.16+n^2\right)^2} \quad (m > 0.4) \qquad (29.11)$$

where

$$\frac{KL_u}{r} < 34 - \left(\frac{12M}{M_2}\right.$$

Table 29.7 gives lateral force factors and wall bottom moment factors which are calculated by above formulas.

29.3.3 Cantilever Retaining Wall Design Example

The cantilever wall is the most commonly used retaining structure. It has a good cost-efficiency record for walls less than 10 m in height. Figure 29.22a shows a typical cross section of a cantilever retaining wall and Table 29.8 gives the active lateral force and the active moment about bottom of the cantilever retaining wall.

For most cases, the following values can be used as the initial assumptions in the reinforced concrete retaining wall design process.

- $0.4 \le B/H \le 0.8$
- $1/12 \le t_{bot}/H \le 1/8$
- $L_{toe} \cong B/3$
- $t_{top} \ge 300$ mm
- $t_{foot} \ge t_{bot}$

TABLE 29.7 Line Load and Point Load Lateral Force Factors

Line Load Factors			Point Load Factors		
$m = x/H$	(f)[a]	(m)[b]	$m = x/H$	(f)[c]	(m)[d]
0.40	0.548	0.335	0.40	0.788	0.466
0.50	0.510	0.287	0.50	0.597	0.316
0.60	0.469	0.245	0.60	0.458	0.220
0.70	0.429	0.211	0.70	0.356	0.157
0.80	0.390	0.182	0.80	0.279	0.114
0.90	0.353	0.158	0.90	0.220	0.085
1.00	0.320	0.138	1.00	0.175	0.064
1.50	0.197	0.076	1.50	0.061	0.019
2.00	0.128	0.047	2.00	0.025	0.007

Notes:
[a] Total lateral force along the length of wall = factor$(f) \times \omega$ (force)/(unit length).
[b] Total moment along the length of wall = factor$(m) \times \omega \times H$ (force × length)/(unit length) (at bottom of footing).
[c] Total lateral force along the length of wall = factor$(f) \times V/H$ (force)/(unit length).
[d] Total moment along the length of wall = factor$(m) \times V$ (force × length)/(unit length) (at bottom of footing).

Example

Given
A reinforced concrete retaining wall as shown in Figure 29.22b:

$H_o = 3.0$ m; surcharge $\omega = 11.00$ kPa
Earth internal friction angle $\phi = 30°$
Earth unit weight $\gamma = 1.8$ *ton/m*^{3}
Bearing capacity $[\sigma] = 190$ kPa
Friction coefficient $f = 0.30$

Solution

1. **Select Control Dimensions**
   Try $h = 1.5$ m, therefore, $H = H_o + h = 3.0 + 1.5 = 4.5$ m.
   Use

$$t_{\text{bot}} = 1/10H = 0.45 \text{ m} \Rightarrow 500 \text{ mm}; \; t_{top} = t_{bot} = 500 \text{ mm}$$

$$t_{\text{foot}} = 600 \text{ mm}$$

   Use

$$B = 0.6H = 2.70 \text{ m} \Rightarrow 2700 \text{ mm};$$

$$L_{\text{toe}} = 900 \text{ mm; therefore, } L_{\text{heel}} = 2.7 - 0.9 - 0.5 = 1.3 \text{ m} = 1300 \text{ mm}$$

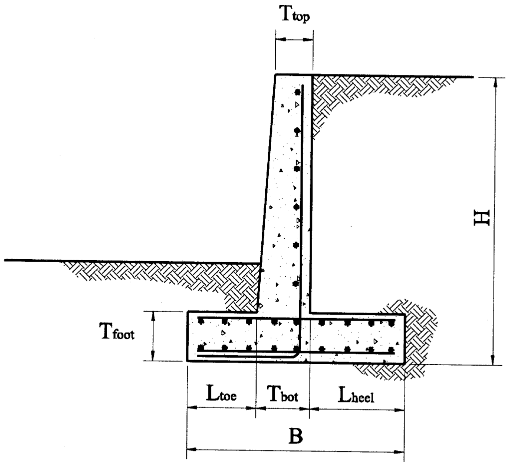
2. **Calculate Lateral Earth Pressure**
   From Table 29.4, $k_a = 0.33$ and $k_p = 3.00$.
   Active Earth pressure:

$$\text{Part 1 (surcharge) } P_1 = k_a \omega H = 0.33(11.0)(4.5) = 16.34 \text{ kN}$$

$$\text{Part 2 } P_2 = 0.5\, k_a \gamma\, H^2 = 0.5(0.33)(17.66)(4.5)^2 = 59.01 \text{ kN}$$

   Maximum possible passive Earth pressure:

$$P_p = 0.5 k_p \gamma h^2 = 0.5(3.00)(17.66)(1.5)^2 = 59.60 \text{ kN}$$



(a)

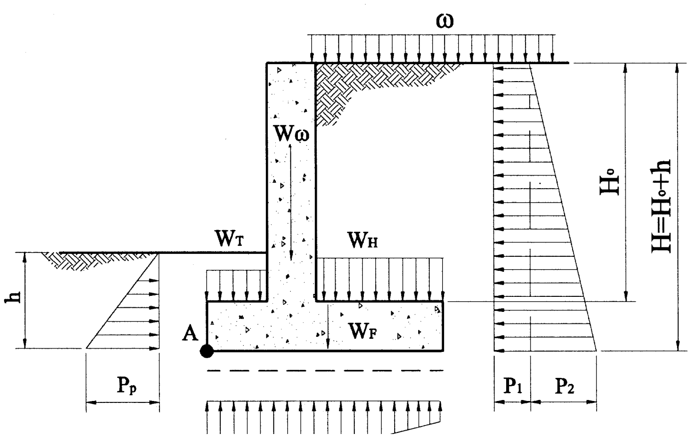


FIGURE 29.22 Design example.

3. Calculate Vertical Loads

Surcharge	W_s (11.00)(1.3)	= 14.30 kN

Use ρ = 2.50 ton/m^3 as the unit weight of reinforced concrete

Wall	W_w 0.50 (4.5 – 0.6) (24.53)	= 47.83 kN
Footing	W_f 0.60 (2.70) (24.53)	= 39.74 kN
Soil cover at toe	W_t 17.66 (1.50 – 0.60) (0.90)	= 14.30 kN
Soil cover at heel	W_h 17.66 (4.50 – 0.60) (1.30)	= 89.54 kN
		Total 205.71 kN

TABLE 29.8 Cantilever Retaining Wall Design Data with Uniformly Distributed Surcharge Load

s	h	1.0	1.2	1.4	1.6	1.8	2.0	2.2	2.4	2.6	2.8	3.0
	p	2.94	4.24	5.77	7.53	9.53	11.77	14.24	16.94	19.89	23.06	26.48
0.00	y	0.33	0.40	0.47	0.53	0.60	0.67	0.73	0.80	0.87	0.93	1.00
	m	0.98	1.69	2.69	4.02	5.72	7.84	10.44	13.56	17.24	21.53	26.48
	p	5.30	7.06	9.06	11.30	13.77	16.47	19.42	22.59	26.01	29.65	33.54
0.40	y	0.41	0.48	0.55	0.62	0.69	0.76	0.83	0.90	0.97	1.04	1.11
	m	2.16	3.39	5.00	7.03	9.53	12.55	16.14	20.33	25.19	30.75	37.07
	p	6.47	8.47	10.71	13.18	15.89	18.83	22.00	25.42	29.06	32.95	37.07
0.60	y	0.42	0.50	0.57	0.65	0.72	0.79	0.86	0.93	1.00	1.07	1.14
	m	2.75	4.24	6.15	8.54	11.44	14.91	18.98	23.72	29.17	35.36	42.36
	p	7.65	9.88	12.36	15.06	18.00	21.18	24.59	28.24	32.12	36.24	40.60
0.80	y	0.44	0.51	0.59	0.67	0.74	0.81	0.89	0.96	1.03	1.10	1.17
	m	3.33	5.08	7.30	10.04	13.34	17.26	21.83	27.11	33.14	39.98	47.66
	p	8.83	11.30	14.00	16.94	20.12	23.53	27.18	31.07	35.18	39.54	44.13
1.00	y	0.44	0.53	0.60	0.68	0.76	0.83	0.91	0.98	1.06	1.13	1.20
	m	3.92	5.93	8.46	11.55	15.25	19.61	24.68	30.50	37.12	44.59	52.95
	p	11.77	14.83	18.12	21.65	25.42	29.42	33.65	38.13	42.83	47.77	52.95
1.50	y	0.46	0.54	0.63	0.71	0.79	0.87	0.94	1.02	1.10	1.17	1.25
	m	5.39	8.05	11.34	15.31	20.02	25.50	31.80	38.97	47.06	56.12	66.19
	p	14.71	18.36	22.24	26.36	30.71	35.30	40.13	45.19	50.48	56.01	61.78
2.00	y	0.47	0.55	0.64	0.72	0.81	0.89	0.97	1.05	1.13	1.21	1.29
	m	6.86	10.17	14.22	19.08	24.78	31.38	38.92	47.45	57.01	67.65	79.43
s	**h**	**3.2**	**3.4**	**3.6**	**3.8**	**4.0**	**4.2**	**4.4**	**4.6**	**4.8**	**5.0**	**5.2**
	p	30.12	34.01	38.13	42.48	47.07	51.89	56.95	62.25	67.78	73.55	79.55
0.00	y	1.07	1.13	1.20	1.27	1.33	1.40	1.47	1.53	1.60	1.67	1.73
	m	32.13	38.54	45.75	53.81	62.76	72.65	83.53	95.45	108.45	122.58	137.88
	p	37.66	42.01	46.60	51.42	56.48	61.78	67.31	73.07	79.08	85.31	91.78
0.40	y	1.17	1.24	1.31	1.38	1.44	1.51	1.58	1.65	1.71	1.78	1.85
	m	44.18	52.14	61.00	70.80	81.59	93.41	106.31	120.35	135.56	151.99	169.70
	p	41.42	46.01	50.83	55.89	61.19	66.72	72.49	78.49	84.72	91.20	97.90
0.60	y	1.21	1.28	1.35	1.42	1.49	1.56	1.62	1.69	1.76	1.83	1.90
	m	50.21	58.95	68.63	79.30	91.00	103.79	117.70	132.80	149.11	166.70	185.61
	p	45.19	50.01	55.07	60.37	65.90	71.66	77.66	83.90	90.37	97.08	104.02
0.80	y	1.24	1.31	1.38	1.45	1.52	1.59	1.66	1.73	1.80	1.87	1.94
	m	56.23	65.75	76.25	87.79	100.41	114.17	129.09	145.25	162.67	181.41	201.52
	p	48.95	54.01	59.31	64.84	70.60	76.60	82.84	89.31	96.02	102.96	110.14
1.00	y	1.27	1.34	1.41	1.49	1.56	1.63	1.70	1.77	1.84	1.90	1.97
	m	62.26	72.55	83.88	96.29	109.83	124.54	140.48	157.70	176.23	196.12	217.43
	p	58.37	64.01	69.90	76.02	82.37	88.96	95.79	102.85	110.14	117.67	125.44
1.50	y	1.32	1.40	1.47	1.55	1.62	1.69	1.76	1.84	1.91	1.98	2.05
	m	77.32	89.55	102.94	117.53	133.36	150.49	168.96	188.82	210.12	232.89	257.20
	p	67.78	74.02	80.49	87.19	94.14	101.32	108.73	116.38	124.26	132.38	140.74
2.00	y	1.36	1.44	1.52	1.59	1.67	1.74	1.82	1.89	1.96	2.04	2.11
	m	92.38	106.56	122.00	138.77	156.90	176.44	197.44	219.94	244.00	269.67	296.97

TABLE 29.8 Cantilever Retaining Wall Design Data with Uniformly Distributed Surcharge Load

s	h	5.4	5.6	5.8	6.0	6.2	6.4	6.6	6.8	7.0	7.2	7.4
	p	85.78	92.25	98.96	105.90	113.08	120.50	128.14	136.03	144.15	152.50	161.09
0.00	y	1.80	1.87	1.93	2.00	2.07	2.13	2.20	2.27	2.33	2.40	2.47
	m	154.41	172.21	191.33	211.81	233.70	257.06	281.92	308.33	336.35	366.01	397.36
	p	98.49	105.43	112.61	120.03	127.67	135.56	143.68	152.03	160.62	169.45	178.51
0.40	y	1.92	1.98	2.05	2.12	2.18	2.25	2.32	2.39	2.45	2.52	2.59
	m	188.72	209.11	230.91	254.17	278.94	305.26	333.18	362.74	394.01	427.01	461.80
	p	104.85	112.02	119.44	127.09	134.97	143.09	151.44	160.03	168.86	177.92	187.22
0.60	y	1.96	2.03	2.10	2.17	2.23	2.30	2.37	2.44	2.50	2.57	2.64
	m	205.88	227.56	250.70	275.35	301.55	329.36	358.81	389.95	422.83	457.51	494.02
	p	111.20	118.61	126.26	134.15	142.27	150.62	159.21	168.04	177.10	186.39	195.92
0.80	y	2.01	2.07	2.14	2.21	2.28	2.35	2.41	2.48	2.55	2.62	2.69
	m	223.04	246.01	270.50	296.53	324.17	353.46	384.43	417.16	451.66	488.01	526.24
	p	117.55	125.20	133.09	141.21	149.56	158.15	166.98	176.04	185.33	194.86	204.63
1.00	y	2.04	2.11	2.18	2.25	2.32	2.39	2.46	2.52	2.59	2.66	2.73
	m	240.19	264.46	290.29	317.71	346.79	377.55	410.06	444.36	480.49	518.51	558.46
	p	133.44	141.68	150.15	158.86	167.80	176.98	186.39	196.04	205.93	216.05	226.40
1.50	y	2.12	2.19	2.26	2.33	2.40	2.47	2.54	2.61	2.68	2.75	2.82
	m	283.08	310.59	339.77	370.67	403.33	437.80	474.14	512.38	552.57	594.76	639.00
	p	149.33	158.15	167.21	176.51	186.04	195.81	205.81	216.05	226.52	237.23	248.17
2.00	y	2.18	2.26	2.33	2.40	2.47	2.54	2.62	2.69	2.76	2.83	2.90
	m	325.97	356.72	389.25	423.62	459.87	498.05	538.21	580.39	624.64	671.01	719.55

s	h	7.6	7.8	7.0	8.2	8.4	8.6	8.8	9.0	9.2	9.5	10.0
	p	169.92	178.98	144.15	197.81	207.57	217.58	227.81	238.29	248.99	265.50	294.18
0.00	y	2.53	2.60	2.33	2.73	2.80	2.87	2.93	3.00	3.07	3.17	3.33
	m	430.46	465.35	336.35	540.67	581.21	623.72	668.25	714.86	763.58	840.74	980.60
	p	187.80	197.34	160.62	217.10	227.34	237.82	248.52	259.47	270.65	287.86	317.71
0.40	y	2.65	2.72	2.45	2.85	2.92	2.99	3.06	3.12	3.19	3.29	3.46
	m	498.43	536.94	394.01	619.79	664.23	710.75	759.38	810.17	863.18	946.94	1098.27
	p	196.75	206.51	168.86	226.75	237.23	247.93	258.88	270.06	281.47	299.03	329.48
0.60	y	2.71	2.77	2.50	2.91	2.98	3.04	3.11	3.18	3.24	3.34	3.51
	m	532.41	572.73	422.83	659.36	705.75	754.26	804.94	857.83	912.98	1000.04	1157.11
	p	205.69	215.69	177.10	236.40	247.11	258.05	269.23	280.65	292.30	310.21	341.25
0.80	y	2.75	2.82	2.55	2.96	3.02	3.09	3.16	3.23	3.29	3.39	3.56
	m	566.39	608.53	451.66	698.92	747.26	797.78	850.50	905.49	962.78	1053.14	1215.94
	p	214.63	224.87	185.33	246.05	257.00	268.17	279.59	291.24	303.12	321.39	353.02
1.00	y	2.80	2.87	2.59	3.00	3.07	3.14	3.20	3.27	3.34	3.44	3.61
	m	600.38	644.32	480.49	738.48	788.78	841.29	896.06	953.14	1012.58	1106.24	1274.78
	p	236.99	247.82	205.93	270.17	281.71	293.47	305.48	317.71	330.19	349.34	382.43
1.50	y	2.89	2.96	2.68	3.10	3.17	3.24	3.31	3.38	3.44	3.55	3.72
	m	685.34	733.81	552.57	837.38	892.57	950.08	1009.97	1072.29	1137.07	1238.99	1421.87
	p	259.35	270.76	226.52	294.30	306.42	318.77	331.36	344.19	357.25	377.29	411.85
2.00	y	2.97	3.04	2.76	3.18	3.25	3.32	3.39	3.46	3.53	3.64	3.81
	m	770.30	823.30	624.64	936.28	996.35	1058.87	1123.88	1191.43	1261.57	1371.74	1568.96

Notes:
1. s = equivalent soil thickness for uniformly distributed surcharge load (m).
2. h = wall height (m); the distance from bottom of the footing to top of the wall.
3. Assume soil density = 2.0 ton/m^3.
4. Active earth pressure factor k_a = 0.30.

Hence, the maximum possible friction force at bottom of footing

$$F = f\,N_{tot} = 0.30\ (205.71) = 61.71\ \text{kN}$$

4. **Check Sliding**
   Total lateral active force (include surcharge)

$$P_1 + P_2 = 16.34 + 59.01 = 75.35\ \text{kN}$$

   Total maximum possible sliding resistant capacity

$$\text{Passive} + \text{friction} = 59.60 + 61.71 = 121.31\ \text{kN}$$

   Sliding safety factor = 121.31/75.35 = 1.61 > 1.50 OK

5. **Check Overturning**
   Take point A as the reference point
   Resistant moment (do not include passive force for conservative)

Surcharge	14.30 (1.3/2 + 0.5 + 0.9)	= 29.32 kN·m
Soil cover at heel	89.54 (1.3/2 + 0.5 + 0.9)	= 183.56 kN·m
Wall	47.83 (0.5/2 + 0.9)	= 55.00 kN·m
Soil cover at toe	14.30 (0.9/2)	= 6.44 kN·m
Footing	39.74 (2.7/2)	= 53.65 kN·m
		Total 327.97 kN·m

   Overturning moment

$$P_1(H/2) + P_2(H/3) = 16.34\ (4.5)/2 + 59.01\ (4.5)/3 = 125.28\ \text{kN·m}$$

Sliding safety factor = 327.97/125.28 = 2.62 > 1.50 OK

6. **Check Bearing**
   Total vertical load

$$N_{tot} = 205.71\ \text{kN}$$

   Total moment about center line of footing:
   - Clockwise (do not include passive force for conservative)

Surcharge	14.30 (2.70/2 – 1.30/2)	= 10.01 kN·m
Soil cover @ heel	89.54 (2.70/2 – 1.30/2)	= 62.68 kN·m
		72.69 kN·m

   - Counterclockwise

Wall	47.83 (2.70/2 – 0.9 – 0.5/2)	= 9.57 kN·m
Soil cover at toe	14.30 (2.70/2 – 0.9/2)	= 12.87 kN·m
Active earth pressure		= 125.28 kN·m
		147.72 kN·m

Total moment at bottom of footing

$$M_{tot} = 147.72\text{-}72.69 = 75.03 \text{ kN·m (counterclockwise)}$$

Maximum bearing stress

$$\sigma = N_{tot}/A \pm M_{tot}/S$$

where
A = 2.70 (1.0) = 2.70 m^2
S = 1.0 (2.7^2) /6 = 1.22 m^3

Therefore:

$$\sigma_{max} = 205.71/2.70 + 75.03/1.22 = 137.69 \text{ kPa}$$
$$< [\sigma] = 190 \text{ kPa}$$

and

$$\sigma_{min} = 205.71/2.70 - 75.03/1.22 = 14.69 \text{ kPa}$$
$$> 0 \qquad \text{OK}$$

7. **Flexure and Shear Strength**
Both wall and footing sections need to be designed to have enough flexure and shear capacity.

29.3.4 Tieback Wall

The tieback wall is the proper structure type for cut sections. The tiebacks are prestressed anchor cables that are used to resist the lateral soil pressure. Compared with other types of retaining structures, the tieback wall has the least lateral deflection. Figure 29.23 shows the typical components and the basic lateral soil pressure distribution on a tieback wall.

The vertical spacing of tiebacks should be between 1.5 and 2.0 m to satisfy the required clearance for construction equipment. The slope angle of drilled holes should be 10 to 15° for grouting convenience. To minimize group effects, the spacing between the tiebacks should be greater than three times the tieback hole diameter, or 1.5 m minimum.

The bond strength for tieback design depends on factors such as installation technique, hole diameter, etc. For preliminary estimates, an ultimate bound strength of 90 to 100 kPa may be assumed. Based on construction experience, most tieback hole diameters are between 150 and 300 mm, and the tieback design capacity is in the range of 150 to 250 kN. Therefore, the corresponding lateral spacing of the tieback will be 2.0 to 3.0 m. The final tieback capacity must be proof-tested by stressing the test tieback at the construction site.

A tieback wall is built from the top down in cut sections. The wall details consist of a base layer and face layer. The base layer may be constructed by using vertical soldier piles with timber or concrete lagging between piles acting as a temporary wall. Then, a final cast-in-place reinforced-concrete layer will be constructed as the finishing layer of the wall. Another type of base layer that has been used effectively is cast-in-place "shotcrete" walls.

29.3.5 Reinforced Earth-Retaining Structure

The reinforced earth-retaining structure can be used in fill sections only. There is no practical height limit for this retaining system, but there will be a certain amount of lateral movement. The essential

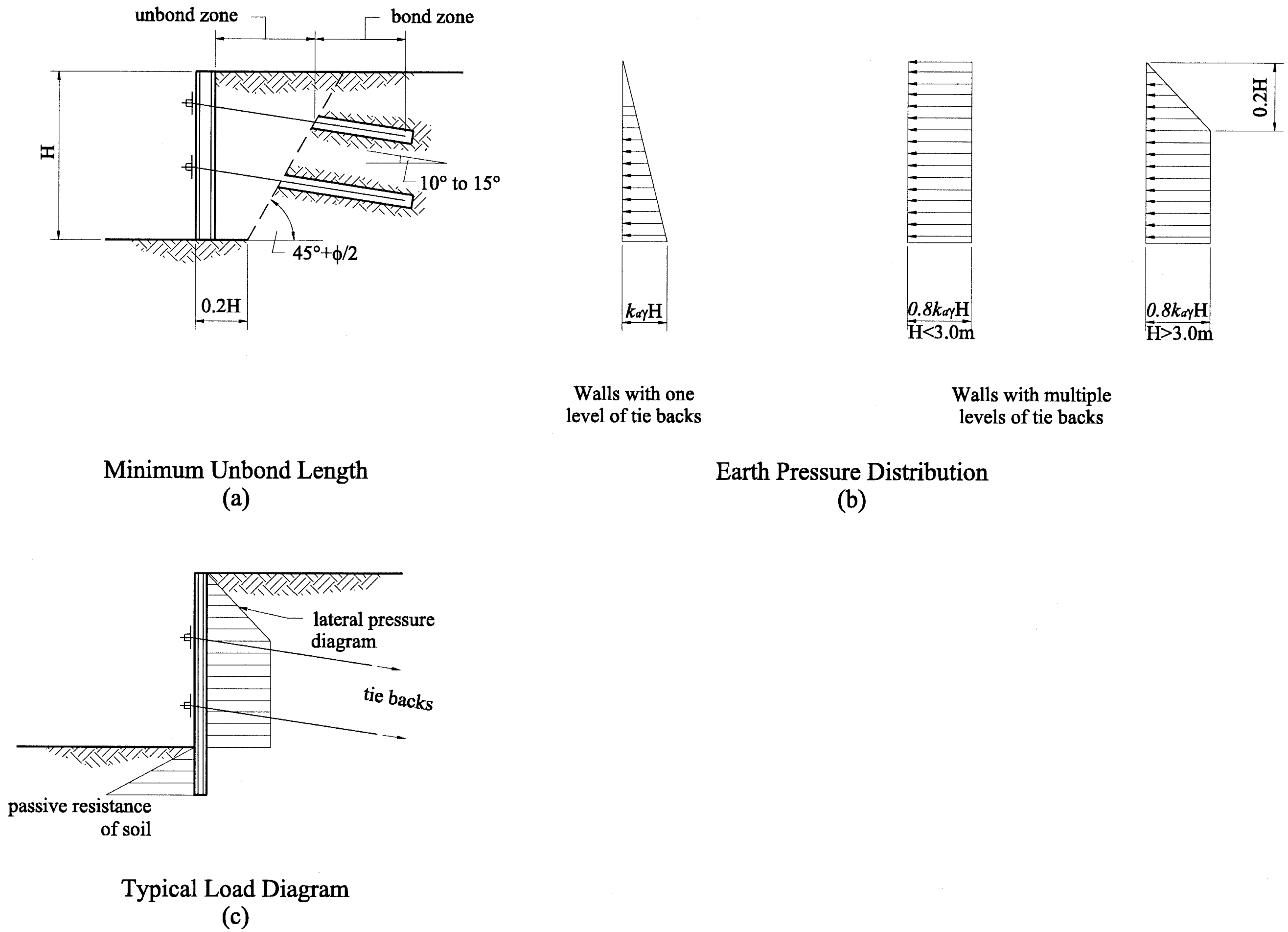


FIGURE 29.23 Tieback wall. (a) Minimum unbond length; (b) earth pressure distribution distribution; (c) typical load diagram.

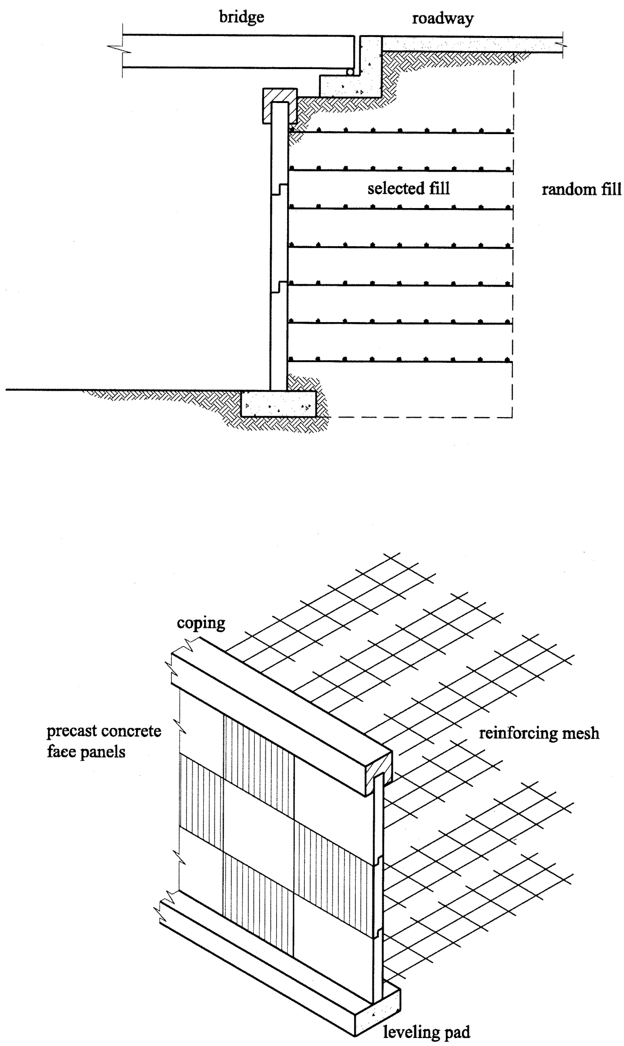


FIGURE 29.24 Mechanical Stabilized Earth (MSE).

concept is the use of multiple-layer strips or fibers to reinforce the fill material in the lateral direction so that the integrated fill material will act as a gravity retaining structure. Figure 29.24 shows the typical details of the MSE retaining structure.

Typically, the width of fill and the length of strips perpendicular to the wall face are on the order of 0.8 of the fill height. The effective life of the material used for the reinforcing must be considered. Metals or nondegradable fabrics are preferred.

Overturning and sliding need to be checked under the assumption that the reinforced soil body acts as a gravity retaining wall. The fiber strength and the friction effects between strip and fill material also need to be checked. Finally, the face panel needs to be designed as a slab which is anchored by the strips and subjected to lateral soil pressure.

29.3.6 Seismic Considerations for Retaining Structures

Seismic effects can be neglected in most retaining structure designs. For oversized retaining structures ($H > 10$ m), the seismic load on a retaining structure can be estimated by using the Mononobe–Okabe solution.

Soil Body ARS Factors

The factors k_v and k_h represent the maximum possible soil body acceleration values under seismic effects in the vertical and horizontal directions, respectively. Similar to other seismic load representations, the acceleration due to gravity will be used as the basic unit of k_v and k_h.

Unless a specific site study report is available, the maximum horizontal ARS value multiplied by 0.50 can be used as the k_h design value. Similarly, k_v will be equal to 0.5 times the maximum vertical ARS value. If the vertical ARS curve is not available, k_v can be assigned a value from $0.1k_h$ to $0.3k_h$.

Earth Pressure with Seismic Effects

Figure 29.25 shows the basic loading diagram for earth pressure with seismic effects. Similar to a static load calculation, the active force per unit length of wall (P_{ac}) can be determined as:

$$P_{ae} = \frac{1}{2} k_{ae} \gamma \left(1 - k_v\right) H^2 \tag{29.12}$$

where

$$\theta' = \tan^{-1}\left[\frac{k_h}{1 - k_v}\right] \tag{29.13}$$

$$k_{ae} = \frac{\sin^2(\phi + \beta - \theta')}{\cos\theta' \sin^2\beta \sin(\beta - \theta' - \delta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\sin(\phi - \theta' - \alpha)}{\sin(\beta - \theta' - \delta)\sin(\alpha + \beta)}}\right]^2} \tag{29.14}$$

Note that with no seismic load, $k_v = k_h = \theta' = 0$. Therefore, $K_{ac} = K_a$.

The resultant total lateral force calculated above does not act at a distance of $H/3$ from the bottom of the wall. The following simplified procedure is often used in design practice:

- Calculate P_{ae} (total active lateral earth pressure per unit length of wall)
- Calculate $P_a = ½\, k_a \gamma H^2$ (static active lateral earth pressure per unit length of wall)

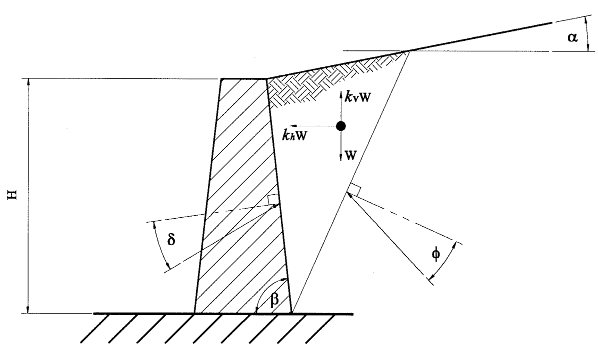


FIGURE 29.25 Load diagram for Earth pressure with seismic effects.

- Calculate $\Delta P = P_{ae} - P_a$
- Assume P_a acts at a distance of $H/3$ from the bottom of the wall
- Assume ΔP acts at a distance of $0.6H$ from the bottom of the wall

The total earth pressure, which includes seismic effects P_{ae}, should always be bigger than the static force P_a. If the calculation results indicate $\Delta P < 0$; use $k_v = 0$.

Using a procedure similar to the active Earth pressure calculation, the passive Earth pressure with seismic effects can be determined as follows:

$$P_{pe} = \frac{1}{2} k_{pe} \gamma \left(1 - k_v\right) H^2 \tag{29.15}$$

where

$$\theta' = \tan^{-1}\left[\frac{k_h}{1-k_v}\right]$$

$$k_{pe} = \frac{\sin^2\left(\beta + \theta' - \phi\right)}{\cos\theta' \sin^2\beta \sin\left(\beta + \theta' + \delta - 90\right)\left[1 - \sqrt{\dfrac{\sin\left(\phi + \delta\right)\sin\left(\phi - \theta' + \alpha\right)}{\sin\left(\beta + \theta' + \delta\right)\sin\left(\alpha + \beta\right)}}\right]^2} \tag{29.16}$$

Note that, with no seismic load, $k_{pc} = k_p$.

References

1. AASHTO, *Standard Specifications for Highway Bridges,* 16th ed., American Association of State Highway and Transportation Officials, Washington, D.C., 1996.
2. *Bridge Memo to Designers Manual*, Department of Transportation, State of California, Sacramento.
3. Brian H. Maroney, Matt Griggs, Eric Vanderbilt, Bruce Kutter, Yuk H. Chai and Karl Romstad, Experimental measurements of bridge abutment behavior, in *Proceeding of Second Annual Seismic Research Workshop,* Division of Structures, Department of Transportation, Sacramento, CA, March 1993.
4. Brian H. Maroney and Yuk H. Chai, Bridge abutment stiffness and strength under earthquake loadings, in *Proceedings of the Second International Workshop of Seismic Design and Retroffitting of Reinforced Concrete Bridges,* Queenstown, New Zealand, August 1994.
5. Rakesh K. Goel, Earthquake behavior of bridge with integral abutment, in *Proceedings of the National Seismic Conference on Bridges and Highways,* Sacramento, CA, July 1997.
6. E. C. Sorensen, Nonlinear soil-structure interaction analysis of a 2-span bridge on soft clay foundation, in *Proceedings of the National Seismic Conference on Bridges and Highways,* Sacramento, CA, July 1997.
7. AEAR, *Manual for Railway Engineering,* 1996.
8. Braja M. Das, *Principles of Foundation Engineering*, PWS-KENT Publishing Company, Boston, MA, 1990.
9. T. William Lambe and Robert V. Whitman, *Soil Mechanics*, John Wiley & Sons, New York, 1969.
10. Gregory P. Tschebotarioff, *Foundations, Retaining and Earth Structures,* 4th ed., McGraw-Hill, New York, 1973.
11. Joseph E. Bowles, *Foundation Analysis and Design,* McGraw-Hill, New York, 1988.
12. Whitney Clark Huntington, *Earth Pressure and Retaining Walls,* John Wiley & Sons, New York.