CHAPTER 11.2 EARTH RETAINING SYSTEMS

TABLE OF CONTENTS

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11.2.1 INTRODUCTION

Earth retaining systems (ERS) are used when construction of a stable slope to maintain elevation cannot be achieved due to insufficient spaces or environmental concerns. ERS can be built for various applications, such as on waterfronts to separate land and water. They can also be used to improve a marginally stable slope (factor of safety greater than 1.0 but does not meet the required value) or to repair a failed slope. ERS are also referred to as earth retention structures or retaining walls. This chapter focuses on information mostly needed by the Structural Designer of the ERS. Communication between the Structural Designer and Geotechnical Designer is of great importance during the design process.

11.2.1.1 Classification of ERS

ERS is typically categorized in the following classifications:

- Rigid Gravity and Semi-Gravity (Conventional) Retaining Wall
- Non-gravity Cantilevered Wall
- Anchored Wall
- Mechanically Stabilized Embankment
- Soil Nail Wall
- Prefabricated Modular Wall

The most common ERS for highway applications include:

a. Conventional reinforced concrete retaining walls on spread footings or piles. Falling under the category of semi-gravity retaining walls, these walls usually have a reinforced concrete or masonry block stem attached to a reinforced concrete footing. Examples include Retaining Walls Types 1, 5, and 6 in the Caltrans Standard Plans. Section 11.2.2 presents a design example of such wall types.

b. Solider pile walls. Falling under the category of non-gravity cantilever walls, these walls consist of steel (W or H shape) or concrete piles with timber, steel, or concrete laggings. They can also be sheet piles, tangent piles, or secant piles without laggings. Section 11.2.3 provides design considerations and a design example of a soldier pile wall.

c. Ground anchor walls. Falling under the category of anchored walls, these walls are soldier pile walls employing ground anchors. When the overhead room is limited, concrete diaphragm walls are used with ground anchors without piles. Section 11.2.4 addresses design considerations and a design example of a ground anchor soldier pile wall.

d. Mechanical stabilized embankment/earth walls (MSE). MSE walls are a cost-effective alternative to conventional gravity walls, cantilever concrete retaining walls, and prefabricated modular retaining systems, particularly where substantial settlements are anticipated. MSE walls behave like gravity walls, deriving lateral resistance through the self-weight of the reinforced soil mass behind the facing.

Section 11.2.5 discusses design considerations and a design example of an MSE wall with concrete panels and metal soil reinforcement.

e. Soil nail walls. These walls typically use reinforced concrete facings connected to steel bars embedded in the soil. Soil nail walls are economical and easy to construct when site conditions are appropriate for their use. Section 11.2.6 covers a soil nail wall facing design example.

f. Gabion Walls. Falling under the category of prefabricated modular walls, gabion walls are composed of stacked metal baskets filled with rocks.

g. Crib Walls. Falling under the category of prefabricated modular walls, crib walls are composed of stretchers and headers forming rectangular cells in which granular soil is placed. These walls derive their stability from the self-weight of the headers, stretchers, and backfilled earth materials. The headers and stretchers can be either concrete or timber.

h. Bin Walls. Falling under the category of prefabricated modular walls, bin walls are usually proprietary. Thin-walled steel stringers and stretchers are bolted onto vertical connectors to form compartments in which compacted granular soil is backfilled. Bin walls are less commonly used.

The examples in this chapter reflect the best design practices developed at Caltrans to the best knowledge of the respective authors and reviewers.

11.2.1.2 Type Selection Considerations

The following list includes the main factors to consider during ERS type selection:

- Site conditions (retained soil types, foundation conditions, settlement requirements, liquefaction potential, corrosiveness of soil and environment, environmental constraints, and nearby bodies of water)
- Site constraints (construction equipment deployment, job site accessibility, and traffic staging)
- Utilities (existing, protected in place, and new)
- Potential impact on nearby structures
- Construction sequence (top-down or bottom-up)
- Maintenance requirement
- Potential for future expansion
- Right-of-way
- Construction easement requirements
- Temporary or permanent easement requirements
- Aesthetics (conformity to nearby structures or landscape)
- Cost
- Time constraints (design and construction)
- Service life of ERS (temporary or permanent applications)

11.2.1.3 General Design Theories

11.2.3.1 Soil Load on Retaining Walls

One of the key aspects of designing retaining walls is determining the lateral earth pressure in the retained soil acting on the retaining wall. Lateral earth pressure on a retaining wall depends on several factors, such as soil type, stress history, the magnitude of strain in the soil, and shear strength of the soil. Construction sequence and retaining wall structure stiffness play a role in the magnitude of lateral earth pressure because these factors affect stress history and strain in the retained soil. This subsection briefly describes some basic theories for estimating lateral earth pressure on a retaining wall.

11.2.1.3.2 Soil Shear Strength

In a soil mass, an infinitesimal soil element at any given point is subject to certain normal and shear stresses. The maximum shear stress that can be resisted in a soil element is called soil shear strength and can be determined according to Mohr Coulomb strength function for that soil. The shear strength at a given point is a function of cohesion, the internal friction angle of the soil, and normal stress applied at that point. The Mohr Coulomb shear strength may be written as:

$$\tau = c + \sigma \tan(\phi_f) \qquad (11.2.1.3\text{-}1)$$

where τ is the shear strength (ksi), c (ksi) is cohesion, and ϕ_f is the internal friction angle of the soil. Cohesion and internal friction angle are often called soil strength parameters. When shear strength in a soil element is reached, it is often said that the soil strength is mobilized at that soil element. Figure 11.2.1-1 illustrates a typical Mohr Coulomb shear strength function for soil with cohesion and internal friction angle. In the figure, the circle shows that the shear and normal stresses in the soil element satisfy the Mohr Coulomb shear strength criteria.

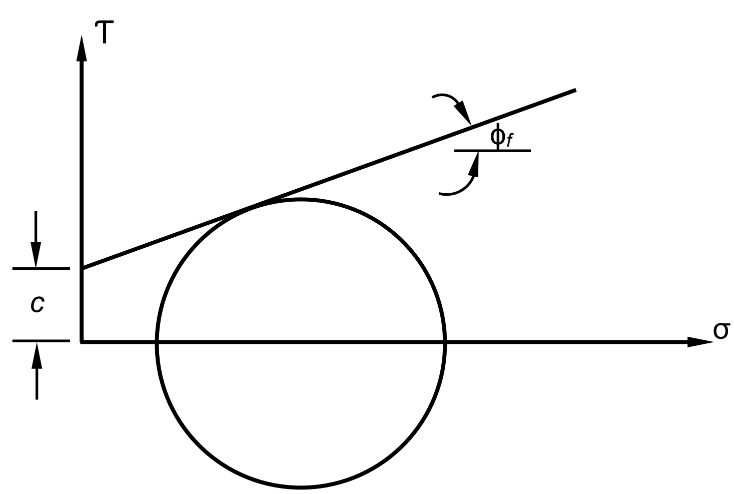


Figure 11.2.1-1 Soil Mohr Coulomb Shear Strength Function

11.2.1.3.3 At-rest Lateral Earth Pressure

If a retaining wall is restrained from moving, the retaining wall is subjected to an “at-rest” lateral earth pressure from the self-weight of the retained soil and any loads applied to the soil. The magnitude of at-rest lateral earth pressure depends on factors such as soil loading history, soil type, etc. At-rest lateral earth pressure can often only be determined empirically.

11.2.1.3.4 Active Lateral Earth Pressure

When a retaining wall moves sufficiently away from retained soil (either by rotation at its base or lateral translation), “slip surfaces” develop in the retained soil. At every point along the slip surface, the shear strength of the retained soil is mobilized. The retained soil is called in the “active state”. Under this condition, the maximum lateral earth pressure on the retaining wall can be determined. The orientation of the slip surfaces can also be determined. The maximum lateral earth pressure is called the active lateral earth pressure. The amount of pressure is usually smaller after a retaining wall moves away from retained soil. Active lateral earth pressure requires only a small movement to develop and is common in unrestrained retaining walls.

There are two main active lateral earth pressure theories utilized in retaining wall design– the Rankine and Coulomb theories.

11.2.1.3.4.1 Rankine Active Lateral Earth Pressure

Rankine theory only applies to soil with a planar ground surface. The lateral earth pressure is evaluated along a vertical plane passing through the end of the heel. The soil shear strength is mobilized at all points in the soil region where the active state develops. The active lateral earth pressure is directly proportional to the soil depth. If there is no surcharge on the soil, the lateral earth pressure is distributed in a triangular shape. A uniform surcharge produces a uniform active lateral earth pressure. The active lateral pressure acts in parallel to the ground surface along the vertical plane. The theory may be applied to the soil with or without cohesion. The Rankine theory cannot predict lateral earth pressure due to a non-uniform surcharge. Slip surfaces predicted by this theory should not be impeded by retaining wall structural components for this theory to be correctly employed. See AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019) Fig C3.11.5.3-1(a) for limitations of the Rankine theory.

11.2.1.3.4.2 Coulomb Active Lateral Earth Pressure

Coulomb theory applies to cohesionless soil with a planar ground surface. At the active state, a planar slip surface is assumed to form in the retained soil, starting from the base of the retained wall and extending to the ground surface. The retained soil bounded by the slip surface and the back of the retaining wall forms a triangular wedge that slides on the retaining wall. It is usually assumed the active lateral earth pressure using this theory is triangularly shaped when there is no surcharge. When a uniform surcharge applies to the ground surface, the induced lateral earth pressure is also uniform. The lateral earth pressure acts on the retaining wall side of the wedge at δ from the direction normal to the side of the retaining wall on which soil wedge slides, and points against soil wedge movement, see Figure 11.2.1-2(a) δ is the friction angle between the soil and retaining wall and is typically assumed to be $2/3\phi$. See AASHTO fig C3.11.5.3-1(b) for conditions where Coulomb theory applies.

Coulomb active lateral earth pressure is numerically identical to Rankine active lateral earth pressure for cohesionless soil with a planar ground surface when the active lateral pressure is evaluated along a vertical plane and setting δ equal to the retained soil ground surface. This is the reason AASHTO allows the use of the Coulomb theory in place where the Rankine theory is traditionally employed.

11.2.1.3.5 Trial Wedge Method

Many times, retained soil neither has a planar surface nor a uniform surcharge. In these cases, Rankine or Coulomb theory may still be used with approximations by employing the trial wedge method. As illustrated in Figure 11.2.1-2(a) for Coulomb condition, the trial wedge method is performed by the following steps:

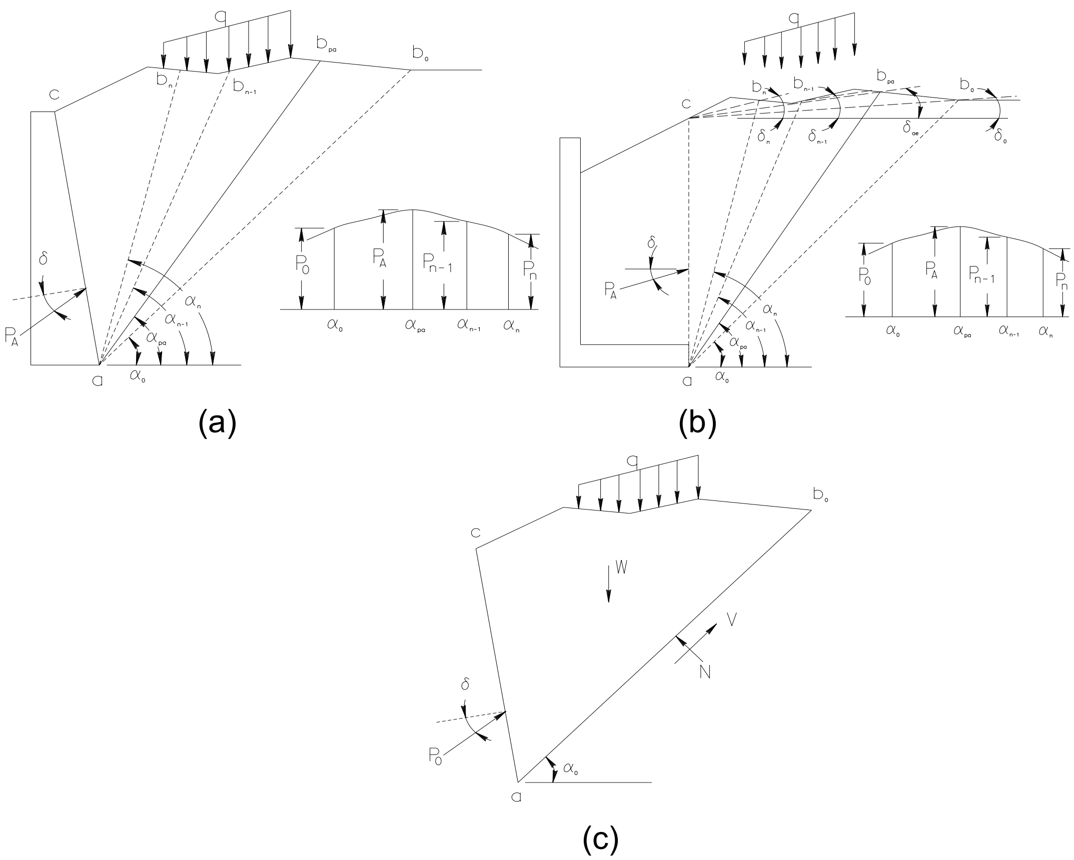


Figure 11.2.1-2 Trial Wedge Method (a) Coulomb Condition; (b) Rankine Condition; (c) Free Body Diagram

1. Assume the slip surface is at inclination α_0 from horizontal in Figure 11.2.1-2(a). Consider the free body diagram of soil mass ab_0c in Figure 11.2.1-2(c). Calculate load, P_0 on Plane *ac*, by balancing the forces on ab_0c in Figure 11.2.1-2(c). Shear force, *V*, and normal force, *N*, on slip surface ab_0, satisfy Mohr Coulomb shear strength function. The orientation of P_0 is at δ from the normal to the retaining wall surface, *ac*.
2. Repeat Step 1 by assuming the slip surface is at a different angle α_2, α_3, …, α_n. Calculate corresponding earth load, P_2, P_3, …, P_n.
3. Plot active earth load as a function of the trial slip surface angles. The maximum value of the active load is the active lateral earth load, P_A., corresponding to the slip surface angle, α_{pa}. The active lateral earth pressure may be assumed to be triangularly shaped when there is no surcharge.

A simplified version of employing the trial wedge method in Rankine condition adopts the above procedures in a similar manner, shown in Figure 11.2.1-2(b). One difference from the Coulomb procedure is the active lateral earth pressure is evaluated on a vertical plane, ac. The other difference is for each trial slip surface. The active earth load is assumed to act at δ from horizontal, which is parallel to a line going through two points on the ground. The first point is where the vertical plane at which the active earth lateral load is evaluated intersects the ground, at c. The other point is where the trial slip surface intercepts the ground. The trial wedge method may account for both irregular retained soil ground and non-uniform surcharge. The active lateral pressure may be assumed to be triangularly distributed when there is no surcharge.

11.2.1.3.6 Passive Lateral Earth Pressure

When a retaining wall moves into the retained soil mass from its original position, lateral earth pressure on the wall increases as the movement increases. At a certain point, any additional movement will not increase the lateral earth pressure on the wall. This is due to the mobilization of the shear strength in the soil near the wall. The retained soil is called at the “passive state” when this happens, and the lateral earth pressure on the wall is called passive lateral earth pressure.

Similar to the active state, there are Rankine and Coulomb theories for estimating passive lateral earth pressure. Rankine theory is only valid when slip surfaces are not impeded by the retaining wall. Rankine theory may greatly underestimate lateral passive earth pressure. Coulomb theory is unconservative when the soil on wall friction angle δ, is more than one-third retained soil friction angle. At larger δ, the slip surface is no longer close to straight, but is a curve near the wall, reducing the amount of lateral earth pressure. Designers should use design charts that are based on log spiral slip surfaces to avoid incorrect estimation of lateral passive earth pressure.

The amount of wall movement required to fully develop a passive state is typically ten times than the required of an active state. Thus, for the strength limit state, only half of the full passive lateral earth pressure may be utilized as resistance.

11.2.1.3.7 Other Lateral Earth Pressure

Lateral earth pressures based on Rankine or Coulomb theories do not apply when calculating pressure on ground anchor walls or on facing of soil nail walls. Lateral earth pressure on those retaining walls is semi-empirical and based on field measurements. Structure components in bin walls, crib walls, or MSEs near the top 20 feet should not be determined by Rankine or Coulomb theories.

In most cases, the lateral earth pressure on ground anchored walls is not triangularly shaped. The lateral earth pressure used in design is called “apparent earth pressure” which represents the maximum pressure the wall experiences.

Coulomb theory is applicable when estimating lateral earth pressure on the back of soil nail walls, MSEs, crib walls, and bin walls for external stability investigation.

11.2.1.3.8 Surcharge Induced Earth Pressure

Surcharge on the level ground of retained soil produces lateral earth pressures on a retaining wall. For a retaining wall restrained from moving, the lateral earth pressure may be determined based on the theory of elasticity, assuming the retained soil as an elastic media. For a flexible retaining wall, the lateral earth pressure may be determined by multiplying the active pressure coefficient by the vertical stress. The vertical stress at a given depth in a flexible wall is approximated to be evenly distributed on a plane at that depth formed by the wall fascia and the base of a frustum with a side slope at 2 vertical to 1 horizontal. See AASHTO Design Specifications for more information.

11.2.1.4 General Design Considerations

All ERS shall be designed to meet requirements at three limit states—service, strength, and extreme event. All ERS shall be designed to achieve external stability, internal stability, overall/compound stability, and other performance-related requirements. The following briefly describes the basic requirements at these three limit states. More information can be found in the current AASHTO LRFD BDS and California Amendments.

11.2.1.4.1 Design Against Foundation Failure (External Stability) and Against Structural Failure (Internal Stability) – Strength and Extreme Event Limit States

Retaining walls shall be designed to resist lateral earth pressure from live and dead loads acting on the retained soil. Retaining walls shall also be designed to resist any loads directly transferred on the structural components. Examples of such cases include vehicle collision loads on barriers attached to retaining walls, loads on sign structures supported on the retaining walls, etc. Retaining walls shall also be designed for lateral earth pressure induced in seismic events by ground motion. Internal drainage systems are generally placed in the retained soil, so water pressure can be disregarded in the design. However, when water in the retained soil cannot be prevented, retaining walls shall be designed for any water pressure. Compaction-induced pressure on retaining walls also needs to be considered either by design against such compaction activity or by using proper construction techniques that do not induce excessive load.

Both foundation (shallow or deep) supporting retaining walls and structural components of the retaining walls shall have adequate strength under imposed loads. The resistance and stability of foundation supporting a retaining wall resisting applied loads is often called external stability. The resistance and stability of structural components of a retaining wall are often called internal stability. In AASHTO LRFD BDS, design for external and internal stability is performed at the strength and extreme event limit states.

Internal stability design - Structural components shall be designed to resist flexure, shear, bearing, tension, compression, torsion, and stability for reinforced concrete, reinforced concrete masonry, and structural steel members. Ground anchors, soil nails and soil reinforcement in MSEs are tension elements and shall be designed not to break or be pulled out under the design tension load.

External stability design - Resistance against sliding, the eccentricity of the reaction force at the footing, and bearing strength on foundation soil shall be adequate in semi-gravity, MSEs, and gravity walls. If the footing on pile foundation is used, the pile foundation shall resist lateral and axial loads transferred through the footing. In the case of a non-gravity cantilever retaining wall, resistance to lateral earth pressure by means of passive resistance of the embedded structural element and resistance against overturning shall be adequate.

11.2.1.4.2 Design for Overall and Compound Stability – Service Limit State

In certain situations, external and internal stability requirements are satisfied, but a critical slip surface may still develop in the soil outside limit of the retaining wall. These include: a) a retaining wall built on a slope, b) several retaining walls built in a tiered manner, or c) a retaining wall is underlain by a weaker soil foundation stratum. Resistance to such failure mode is called the overall or global stability of retaining walls. Other times, the critical slip surface partially intercepts structural elements. Resistance to such failure mode is called compound stability of retaining walls. Both overall and compound stability in a retaining wall shall be satisfied. The generalized limit equilibrium method is often used to check overall and compound stability.

In AASHTO LRFD BDS, overall and compound stability is investigated at the service limit state by geotechnical designers.

11.2.1.4.3 Design for Serviceability – Service Limit State

The following list includes several factors to consider during retaining wall design:

- Prevent excessive settlement, lateral movement, or tilting during its service life. The extent of tolerable movement of the retained or affected facility, critical utility, or structure will determine any allowable movement or deformation.
- Avoid large deformations to prevent negative public perception.
- Excessive differential settlement may damage structural components.

- Account for crack width control of reinforced concrete components, shrinkage, and temperature effects.
- Account for winter climate effects (e.g., corrosive soil and salt spray on structure components, frost on auxiliary components such as internal drainage, freeze-thaw on footings.)
- Account for scour and erosion effects.
- Consider the required space for access for retaining wall inspection and maintenance work.

Another factor involves large water pressure in retained soil. The total pressure on a retaining wall from a water-saturated backfill can be twice as much as the lateral earth pressure from a water-free backfill. Adequate internal drainage in the backfill shall be provided. Surface drainage shall also be provided for the surface runoff above retained soil. In addition, surface runoff drainage and internal drainage should be separated–surface water should not be directed into the internal drainage system.

Temperature and shrinkage effects on retaining walls can be mitigated by providing sufficient bar reinforcement in concrete components or by providing expansion joints and weakened planes in concrete wall stems exposed to the weather.

In AASHTO LRFD BDS, serviceability design is performed at service limit state.

11.2.1.5 Design Processes

Certain retaining walls require following a set of type-specific procedures. However, the general steps for the structural design of retaining walls are as follows:

1. Determine the layout line of the retaining wall. Ascertain details of the finished grade on the front and back of the wall.
2. Divide the retaining wall into segments and represent each segment in terms of loads and geometry.
3. Determine required elevations either at the bottom of the footing or wall to establish a design height.
4. Select trial wall component cross sections and how the components fit together based on past experience.
5. Obtain soil strength parameters such as internal friction angle and cohesion from geotechnical designers. Determine lateral earth pressure based on soil strength parameters, wall geometry, and retained soil. Determine lateral earth pressure on the retaining wall induced by surcharge loads or loads in retained soil. Obtain lateral earth pressure or loads from geotechnical engineers when designing complex walls, such as multi-tier walls, special loading conditions, etc.
6. Calculate loads or pressure on the wall for overall stability. Loads on retaining walls are larger than that determined in Step 5 if the overall stability governs the design. Geotechnical Designers will analyze overall stability and provide the design load on the retaining walls. Revise retaining wall component dimensions if necessary.

7. Check settlement and deflection requirements. Revise retaining wall component dimensions if necessary.
8. Analyze retaining walls at strength and extreme event limit states for external stability. Revise retaining wall dimensions if necessary.
9. Calculate internal loads/forces on structural components. Examples include:
   - Concrete and steel elements: moment, torsional, and shear diagrams
   - Ground anchor walls: tension loads in ground anchors
   - MSEs: loads in connection and soil reinforcement
   - Soil nail walls: nail head loads and soil nail load
   - Pile foundations: lateral and axial loads in piles
   - Soldier piles and sheet piles: embedment depth
10. Design structural components based on the internal forces obtained in the previous step. Revise retaining wall component dimensions if necessary.
11. Design for corrosion protection and for protection in a freeze-thaw environment.
12. Design drainage system for internal drainage in retained soil and surface runoff.
13. Design for temperature and shrinkage effects. Determine locations of expansion joints and weakened plane joints as applicable.

11.2.1.6 Caltrans Design Guidance Publications

Design of retaining walls shall comply with AASHTO LRFD Bridge Design Specifications as amended in the California Amendments. Because the seismic design of retaining walls is currently in development, most of the examples in this chapter do not include seismic design. Several publications, such as the Structure Technical Policies (STPs) and Bridge Design Memos (BDMs), are available to provide guidance to designers on certain topics and have standard designs of retaining walls. Bridge Design Details have ERS-related detailing information. The main ERS-related design guidance publications are:

- STP Section 11 - Walls, Abutments, and Piers
- BDM Section 11 - Walls, Abutments, and Piers
- BDD Chapter 6 - Abutments
- ERS in Standard Plans
- ERS in Bridge Design Details
- Caltrans Geotechnical Manual

11.2.2 REINFORCED CONCRETE RETAINING WALL DESIGN EXAMPLE

11.2.2.1 Design Data

A conventional reinforced concrete retaining wall on spread footing is shown in Figure.11.2.2-1.

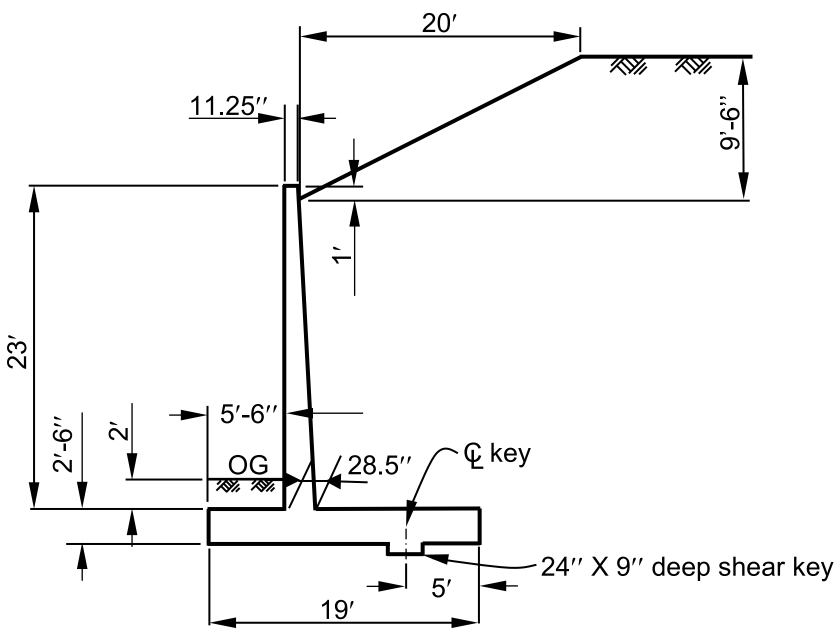


Figure 11.2.2-1 Retaining Wall Supporting Backfill

The level area of the backfill does not support traffic. Groundwater does not exist under the spread footing to affect the retaining wall bearing resistance. Water does not accumulate in the backfill, which assumes adequate internal drainage.

The backfill and foundation soil has the following properties,

Site class is D.

$\gamma_s = 120$ pcf (unit weight)

$\phi_f = 34°$ (soil internal friction angle)

$c = 0$ psf (soil cohesion)

$V_s = 900$ ft/sec (shear wave velocity in the backfill)

$HPGA = 0.7g$ (site adjusted horizontal peak ground acceleration)

$PGV = 39.9$ in./sec (site peak ground velocity)

The spectral ground acceleration is shown in Figure 11.2.2-2. The permissible net bearing stress under service load to limit settlement is shown in Figure 11.2.2-3. The nominal footing bearing resistance is shown in Figure 11.2.2-4.

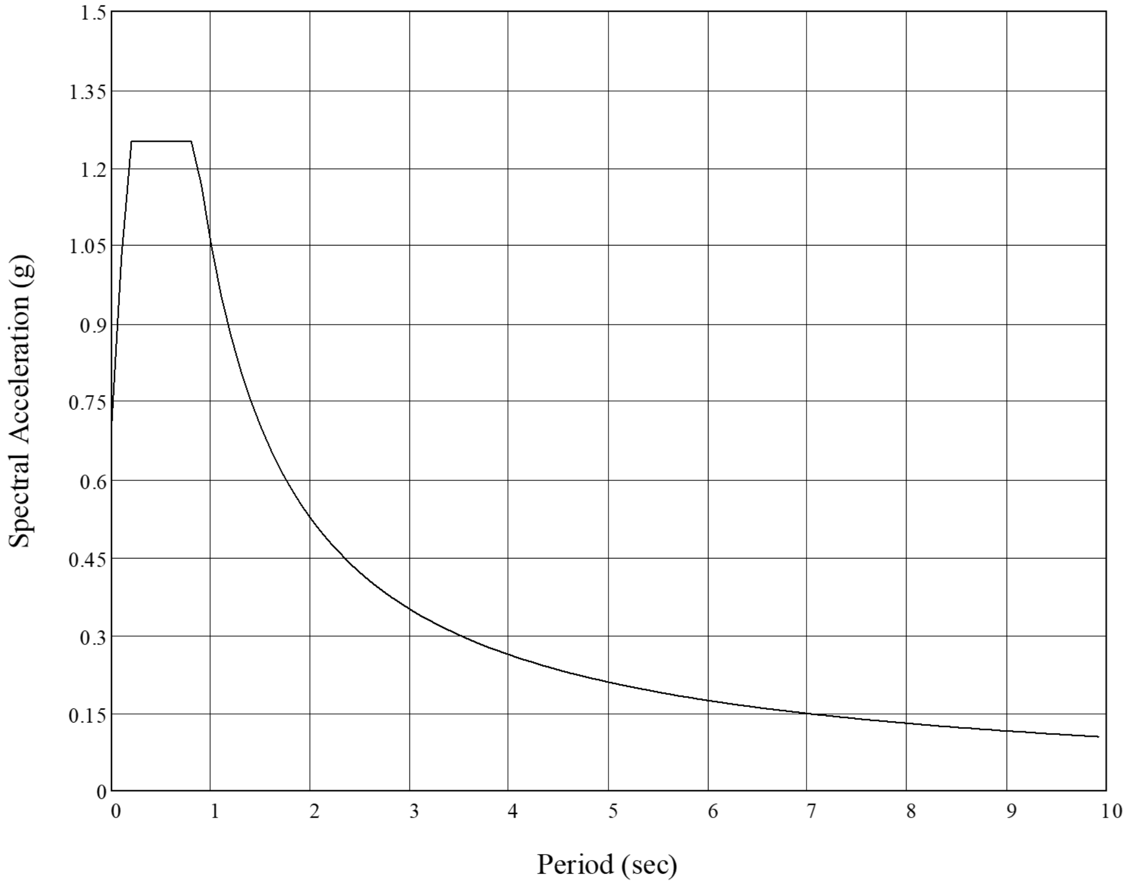


Figure 11.2.2-2 Site Specific ARS Curve

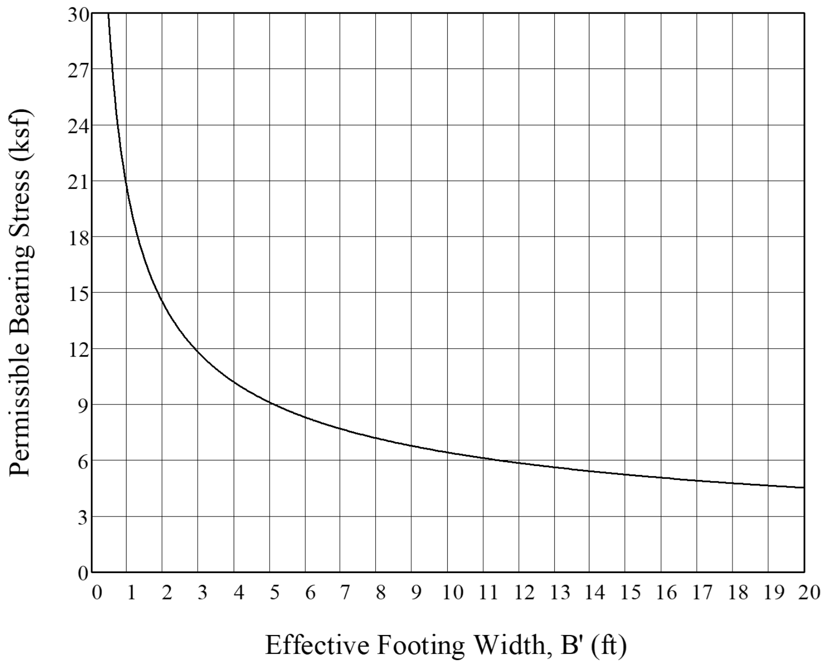


Figure 11.2.2-3 Permissible Net Bearing Stress at Service Limit State

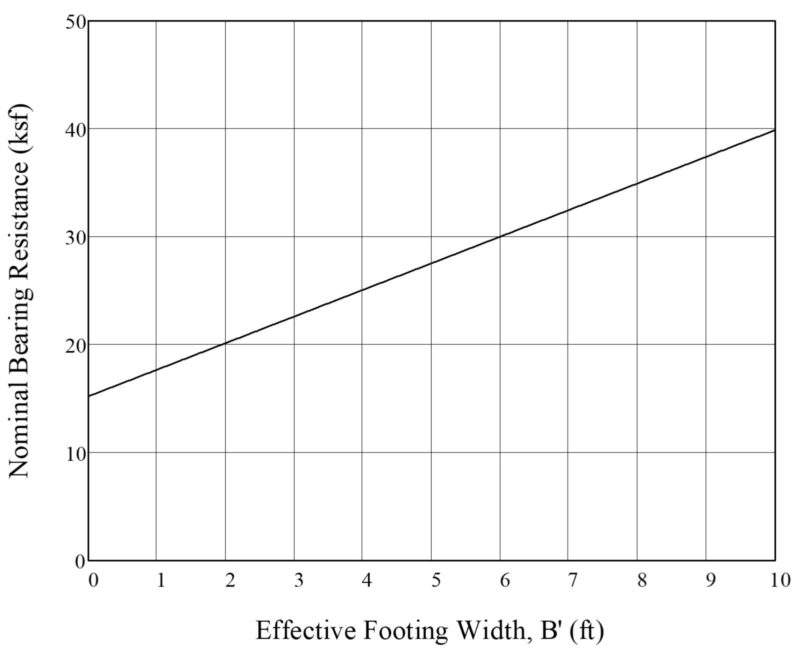


Figure 11.2.2-4 Nominal Footing Bearing Resistance

The reinforced concrete has the following properties,

$$\gamma_c = 150 \text{ pcf (unit weight)}$$

$$f_c^{'} = 4000 \text{ psi}$$

$$f_y = 60000 \text{ psi}$$

11.2.2.2 Design Requirements

Perform the following design in accordance with the AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019) and STP 11.29 (Caltrans, 2020). Assume the wall is “typical”. Use $k_h = 0.28$ for seismic design and do not consider wave scattering effects due to the wall height.

- Check for external stability for the extreme event (seismic), service, and strength limit states
- Design for internal stability (design structure elements) for the extreme event (seismic), service, and strength limit states
- The mean displacement under seismic load shall not exceed 6 inches.

11.2.2.3 Design for External Stability

By inspection, the retaining wall design is governed by seismic loads. Therefore, design can be started from extreme event checks.

11.2.2.3.1 Extreme Event Limit State (Seismic)

According to Figure 11.2.2-2, the horizontal peak ground acceleration (HPGA) is 0.7g. k_h is 0.28 as a design parameter and is less than the HPGA/g. This implies the wall slides at design seismic load.

$$\frac{k_h}{HPGA/g} = \frac{0.28}{0.7} = 0.4$$

$$k_h = 0.4\frac{HPGA}{g}$$

Since $k_h < 0.5\dfrac{HPGA}{g}$, the retaining wall should be designed using the Newmark sliding block method to assess the permanent displacement under seismic loads (AASHTO Article 11.6.5.2.2).

Figure 11.2.2-5 shows the free body diagram of the retaining wall and the forces applied.

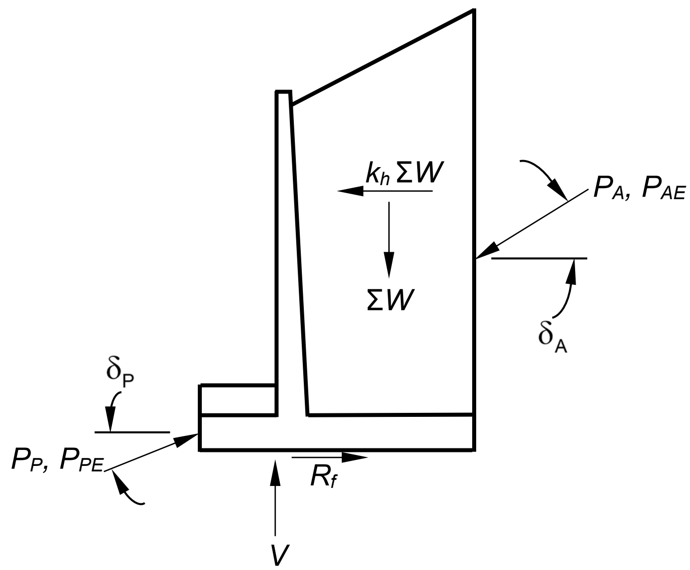


Figure 11.2.2-5 Free Body Diagram of Retaining Wall

- Calculate δ_A, P_A, P_{AE}

  It is assumed that the seismic lateral earth load, P_{AE}, and the static lateral earth load, P_A, act on AB at the same angle, δ_A. The first step is to calculate P_A, and the associated δ_A, P_A, and δ_A are then determined by the trial wedge method (refer to Section 11.2.1.3.5).

  $$\text{AB} = 30.44 \text{ ft}$$

  $$P_A = 17.54 \text{ kip}$$

  $$\delta_A = 9.74^\circ$$

  Use the trial wedge method to determine P_{AE}, given $k_h = 0.28$ and $\delta_A = 9.74^\circ$.

  $$P_{AE} = 31.56 \text{ kip}$$

  P_{AE} is assumed to act at $\frac{1}{3}\text{AB}$ from the bottom of the footing.

- Calculate ΣW

  Break down wall and soil components into parts with simple geometry, as shown in Figure 11.2.2-6. Applied forces are also shown in the figure.

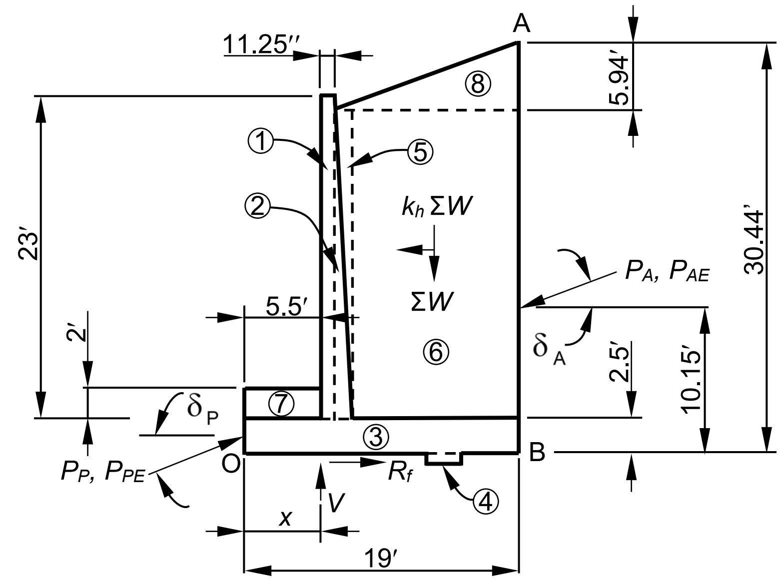


Figure 11.2.2-6 Retaining Wall Parts and Applied Forces

Table 11.2.2-1 shows the calculation of the self-weight of the stem, the footing, and the soil above the footing. The vertical reaction at the bottom of the footing, V,

$$V = \sum W + P_{AE} \sin \delta_A - P_{PE} \sin \delta_P$$

The last term in the above equation is small and is omitted in calculating V.

ΣW is the combined self-weight of the concrete retaining wall and the soil over the footing.

Table 11.2.2-1 Self-weight of Retaining Wall and the Soil above the Footing

Part Number	Weight (kip)	Load Factor	Factored Weight (kip)
1	3.23	1.0	3.23
2	2.48	1.0	2.48
3	7.12	1.0	7.12
4	0.22	1.0	0.22
5	1.82	1.0	1.82
6	29.37	1.0	29.37
7	1.32	1.0	1.32
8	4.45	1.0	4.45
			ΣW = 50.01

- Calculate P_{PE}

It is typically assumed that P_{PE} is applied on the footing and shear key at the angle of $\frac{2}{3}\phi_f$, $\delta_P = \frac{2}{3}\phi_f = 22.67°$

P_{PE} includes passive soil load on the footing and shear key, as shown in Figure 11.2.2-7.

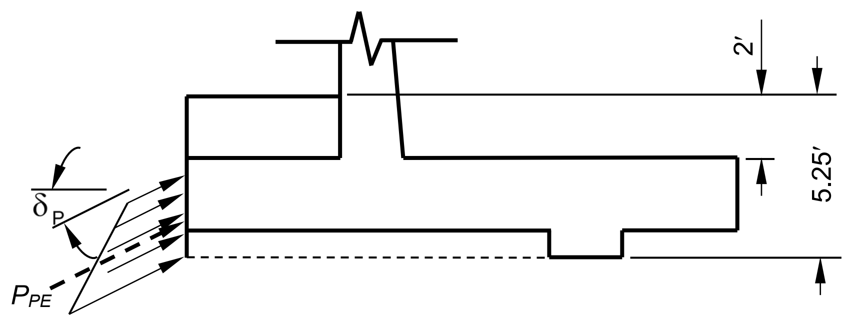


Figure 11.2.2-7 Passive Force on Footing and Shear Key

$$P_{PE} = \frac{1}{2}k_{PE}\gamma_s(5.25^2 - 2^2)$$

From AASHTO Figure A11.4-2, when $k_h = 0.28$,

$$k_{PE}\cos\delta_P = 5.2$$

$$k_{PE} = \frac{5.2}{\cos 22.67°} = 5.64$$

$$P_{PE} = \frac{1}{2}(5.64)(0.12)(5.25^2 - 2^2) = 8.0 \text{ kip}$$

- Check Displacement

According to AASHTO A11.5, seismic displacement can be estimated using the three methods in the section. In this example, we use the method suggested by Anderson (2008).

Anderson Method (AASHTO A11.5.2)

The site category is “D” and in Western Region. AASHTO equation A11.5.2-3 is used.

$$\log(d) = -1.51 - 0.74\log(\frac{k_y}{k_{h0}}) + 3.27\log(1 - \frac{k_y}{k_{h0}}) - 0.80\log(k_{h0}) + 1.59\log(PGV)$$

In this equation k_y is the acceleration of the retaining wall at the onset of sliding, and numerically equal to k_h, since wave scattering effect is not considered, and *PGV* is the peak ground velocity.

k_{h0} is equal to $HPGA/g$. Thus,

$k_y = k_h = 0.28$

$k_{h0} = 0.70$

$PGV = 39.9$ in./sec

$d = 5.4$ in. $<$ allowable displacement $= 6$ in. OK

- Check Bearing Capacity and Eccentricity Requirements

  Figure 11.2.2-5 shows loads applied to the retaining wall. Figure 11.2.2-6 shows the part numbers of the wall components. Table 11.2.2-2 lists all the forces applied to the wall and the moments of the forces about point “O” at the lower left corner of the footing in Figure 11.2.2-5. For an extreme event (seismic) limit state, all load factors are 1.0 for forces and moments. The resistance factor of P_{PE} is also 1.0. The contribution of P_{PE} is again ignored. Thus, the forces and the moments in the table below are both nominal and factored. Let the moments about point “O” be positive in the clockwise direction and negative in the counter-clockwise direction.

Table 11.2.2-2 Forces and Moments about "O" for Extreme Event Limit State (Seismic)

Part/Force I.D.	Force (kip)	Moment Arm (ft)	Moment about "*O*" (kip-ft)
1	3.23(↓)	5.97	19.28
2	2.48(↓)	6.92	17.15
3	7.12(↓)	9.50	67.69
4	0.22(↓)	14.00	3.15
5	1.82(↓)	7.42	13.50
6	29.37(↓)	13.44	394.66
7	1.32(↓)	2.75	3.63
8	4.45(↓)	14.83	66.06
$P_{AE}\sin(\delta_A)$	5.34(↓)	19.0	101.46
$P_{AE}\cos(\delta_A)$	31.11(←)	10.15	-315.77
V	50.02+5.34=55.36(↑)	x	-55.36x
1	0.28(3.23)=0.91(←)	14.0	-12.66
2	0.28(2.48)=0.69(←)	10.17	-7.06
3	0.28(7.12)=2.00(←)	1.25	-2.49
4	0.28(0.22)=0.06	0.38	0.02
5	0.28(1.82)=0.51(←)	17.17	-8.75
6	0.28(29.37)=8.22(←)	13.5	-111.02
7	0.28(1.32)=0.37(←)	3.5	-1.29
8	0.28(4.45)=1.25(←)	26.48	-33.10

Sum the moments in the last column in the above table,

$$\sum M_O = 194.46 - 55.36x$$

$$\therefore x = 3.52 \text{ ft}$$

Eccentricity

The eccentricity, e, is

$$e = \frac{19}{2} - 3.52 = 5.98 \text{ ft}$$

Per CA 3.4.1, γ_{EQ} is taken as 0.0 and per AASHTO 11.6.5.1, the permissible eccentricity, e_r, is

$$e_r = \frac{19}{3} = 6.33 \text{ ft} > e \text{ OK}$$

Bearing Capacity

Effective footing width, B′,

$$B' = B - 2e = 19 - 2(5.98) = 7.03 \text{ ft}$$

$$q_u = \frac{V}{B'} = \frac{55.35}{7.03} = 7.87 \text{ ksf}$$

According to Figure 11.2.2-4, the nominal bearing resistance, q_n, is

$$q_n = 32.22 \text{ ksf}$$

$$\phi_n = 0.8 \qquad \text{(AASHTO 11.5.8)}$$

$$q_r = \phi_n q_n = 0.8(33.22) = 26.58 \text{ ksf}$$

$$q_u < q_r \text{ OK}$$

11.2.2.3.2 Service Limit State

- Check Overall Stability

  This check is performed by a Geotechnical Engineer. The overall stability of a retaining wall is typically adequate when not on a slope or in the absence of a weak soil layer below the retaining wall footing.

- Calculate Forces on the Wall

  Figure 11.2.2-6 shows the retaining wall and soil over the footing that are divided into parts with simple geometry. The figure also shows the forces applied to the wall. In the figure, P_A is active soil force on the back of the wall, *AB*, and may be found using the trial wedge method.

$$P_A = 17.54 \text{ kip}$$

$$\delta_s = 9.74^\circ$$

$$h = 10.15 \text{ ft}$$

$$\delta_P = \frac{2}{3}\phi_f = 22.67^\circ$$

From AASHTO Figure 3.11.5.4-1, with $\phi_f = 34^\circ$ and $\delta_P = 22.67^\circ$,

$k_P = 7.05$

$$P_P = \frac{1}{2}(7.05)(0.12)(5.25^2 - 2^2) = 9.97 \text{ kip}$$

Table 11.2.2-3 lists wall and soil component self-weight, applied forces, and moments about point "O". Vertical component of P_P has little effect on the magnitude of the vertical reaction force, V, and is hence ignored. P_P is also ignored as it has little effect on the moment about point "O".

All loads have load factors equal to 1.0 at the service limit state. Thus, the forces and moments are both nominal and factored values.

Table 11.2.2-3 Forces and Moments about "O" for Service Limit State

Part/Force I.D.	Force (kip)	Moment Arm (ft)	Moment about "O" (kip-ft)
1	3.23(↓)	5.97	19.28
2	2.48(↓)	6.92	17.15
3	7.12(↓)	9.50	67.69
4	0.22(↓)	14.00	3.15
5	1.82(↓)	7.42	13.50
6	29.37(↓)	13.44	394.47
7	1.32(↓)	2.75	3.63
8	4.45(↓)	14.83	65.99
$P_A \sin(\delta_A)$	2.97(↓)	19.0	56.43
$P_A \cos(\delta_A)$	17.29(←)	10.65	-175.49
V	50.01+2.97=52.98(↑)	x	-52.98x

Sum all the moments about point "O" from the last column in the above table,

$M_O = 465.8 - 52.98x$

$x = 8.79$ ft

- Check Settlement:

  Eccentricity, e,

  $$e = \frac{B}{2} - x = \frac{19}{2} - 8.79 = 0.70 \text{ ft}$$

  Effective footing width,

  $B' = B - 2e = 19 - 2(0.70) = 17.59$ ft

Gross factored footing stress, q_u,

$$q_u = \frac{52.98}{17.59} = 3.01\,\text{ksf}$$

The Original Ground (OG) is two feet above the top of the footing. Before the wall is constructed, the original overburden pressure is:

$$q_{O.G} = \gamma D_f = 0.12(4.5) = 0.54 \text{ ksf}$$

The net bearing pressure at the bottom of the footing,

$$q_u^{'} = 3.01 - 0.54 = 2.47 \text{ ksf}$$

According to Figure 11.2.2-3, for B′=17.59 ft, the permissible footing bearing stress for settlement at service,

$$q_{pn} = 4.9\text{ksf} > 2.47 \text{ ksf}$$

Thus, settlement is OK.

11.2.2.3.3 Strength Limit State

- Check Footing Bearing Stress (Strength Limit State Ia)

To obtain the maximum bearing stress, use maximum load factors from CA Table 3.4.1-1,

$\gamma_p = 1.35$ for soil self-weight

$\gamma_p = 1.25$ for concrete self-weight

$\gamma_p = 1.50$ for horizontal active earth pressure

The resistance factor from CA Table 11.5.7-1,

$\phi_r = 0.55$ for bearing resistance

Figure 11.2.2-6 shows the forces applied to the retaining wall. As previously mentioned, P_P has little effect on bearing calculation and is ignored.

Table 11.2.2-4 lists forces and moments about point “O”. The moments are positive in clockwise direction, negative in counterclockwise direction.

Table 11.2.2-4 Forces and Moments about "O" for Strength Limit State Ia

Part/Force I.D.	Force (kip)	Load Factor, γ_P	Factored Load (kip)	Moment Arm (ft)	Factored Moment about "O" (kip-ft)
1	3.23(↓)	1.25	4.04	5.97	24.11
2	2.48(↓)	1.25	3.10	6.92	21.45
3	7.12(↓)	1.25	8.91	9.50	84.61
4	0.22(↓)	1.25	0.28	14.00	3.94
5	1.82(↓)	1.35	2.45	7.42	18.18
6	29.37(↓)	1.35	39.65	13.44	532.90
7	1.32(↓)	1.35	1.78	2.75	4.9
8	4.45(↓)	1.35	6.01	14.83	89.13
$P_A \sin(\delta_A)$	2.97(↓)	1.50	4.45	19.0	84.55
$P_A \cos(\delta\ \ )$	17.29(←)	1.50	25.94	10.15	-263.29
V	-	-	70.68	x	-70.68x

V is the sum of all the vertical factored forces in Column 4 in the table above,

$V = 70.68$ kip

$\sum M_O = 600.48 - 70.68x$

$x = 8.50$ ft

The effective footing width, B′

$B' = 2x = 2(8.50) = 17.00$ ft

Bearing stress at the bottom of the footing,

$$q_u = \frac{70.68}{17.00} = 4.16 \text{ ksf}$$

According to Figure 11.2.2-4, nominal footing bearing resistance,

$q_n = 57.78$ ksf

Factored footing bearing resistance,

$q_r = 0.55(57.78) = 31.78$ ksf

$q_u < q_r$ OK

- Check Eccentricity and Sliding (Strength Limit State Ib)

To achieve maximum eccentricity, use maximum load factors for lateral earth pressure and minimum load factors for vertical loads from CA Table 3.4.1-1,

$\gamma_p = 1.00$ for soil self-weight

$\gamma_p = 0.90$ for concrete self-weight

$\gamma_p = 1.50$ for horizontal active earth pressure

The resistance factor from CA Table 11.5.7-1,

$\varphi_\tau = 1.0$ for friction resistance

$\varphi_{ep} = 0.5$ for passive resistance

Table 11.2.2-5 lists the forces and moments about point "O",

Table 11.2.2-5 Forces and Moments about "O" for Strength Limit State Ib

Part/Force I.D.	Force (kip)	Load Factor, γ_P	Factored Load (kip)	Moment Arm (ft)	Factored Moment about "O" (kip-ft)
1	3.23(↓)	0.9	2.91	5.97	17.37
2	2.48(↓)	0.9	2.23	6.92	15.44
3	7.12(↓)	0.9	6.41	9.50	60.90
4	0.22(↓)	0.9	0.20	14.00	2.80
5	1.82(↓)	1.0	1.82	7.42	13.50
6	29.37(↓)	1.0	29.37	13.44	394.730
7	1.32(↓)	1.0	1.32	2.75	3.63
8	4.45(↓)	1.0	4.45	14.83	65.99
$P_A \sin(\delta_A)$	2.97(↓)	1.50	4.45	19.0	84.55
$P_A \cos(\delta_A)$	17.29(←)	1.50	25.94	10.15	-263.29
V	-	-	53.16	x	-53.16x

V is the sum of all the vertical factored forces in Column 4 in the table above,

$V = 53.16$ kip

Sum all the moments about point "O" from the last column in the above table,

$M_O = 395.62 - 53.16x$

$x = 7.44$ ft

Eccentricity

$$e = \frac{19}{2} - 7.44 = 2.06 \text{ ft}$$

Permissible eccentricity, e_r (AASHTO 11.6.3.3),

$$e_r = \frac{B}{3} = 6.3 \text{ ft} > e \ \text{ OK}$$

Sliding

Horizontal factored active earth load,

$$Q_u = \gamma_p P_A \cos\delta_A = 1.50(17.54)\cos(9.74°) = 25.94 \text{ ksf}$$

Horizontal factored friction resistance,

$$\varphi_\tau V \tan\phi_f = 1.0(53.16)\tan(34°) = 35.86 \text{ kip}$$

Horizontal factored passive resistance,

$$\varphi_{ep} P_P \cos\delta_P = 0.50(9.97)\cos(22.67°) = 4.60 \text{ kip}$$

Total horizontal factored resistance,

$$R_r = 35.86 + 4.60 = 40.5 \text{ kip} > Q_u \ \text{ OK}$$

11.2.2.4 Design for Internal Stability (Structure Element Design)

11.2.2.4.1 Stem

Calculate Internal Forces

Strength Limit State

The load factors from CA Table 3.4.1-1,

$$\gamma_p = 1.50 \text{ for horizontal active earth pressure}$$

The lateral load on the stem results from the horizontal active earth pressure. Soil pressure is obtained by performing a trial wedge analysis along AB; see Figure 11.2.2-8.

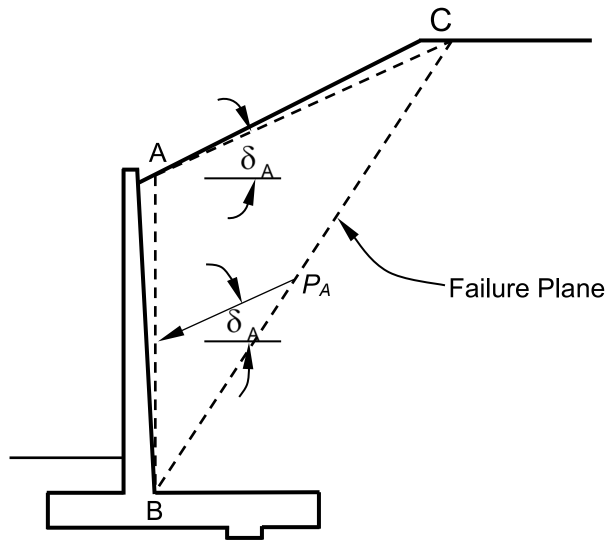


Figure 11.2.2-8 P_A for Stem Design

$$\delta_A = 21.14^\circ$$

$$P_A = 11.42 \text{ kip}$$

P_A is assumed to be applied at $\frac{1}{3}\text{AB}$ from the bottom of the stem.

A free body diagram of the stem is shown in Figure 11.2.2-9. In the figure, W_1 is the self-weight of the stem. W_2 is the self-weight of soil A′AB. N_B is the vertical force on BB′. V_B is the shear at the bottom of the stem. M_B is the moment at the bottom of the stem.

Since the stem batter is small, the axis of the stem is assumed to be vertical, and the weight of the W_2 is ignored when calculating the internal forces in the stem.

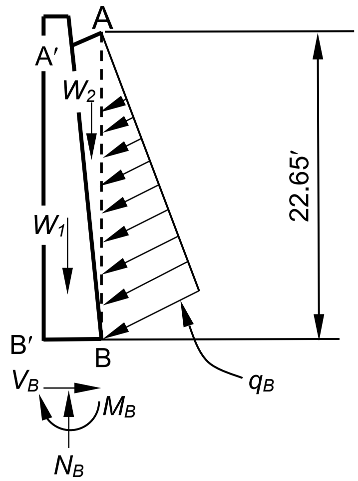


Figure 11.2.2-9 Forces on Stem at Service and Strength Limit State

The earth pressure at Point "B", q_B, is

$$q_B = \frac{2P_A}{H}$$

where H is the length of AB.

$$q_B = \frac{2(11.42)}{22.653} = 1.01 \text{ ksf}$$

- Calculate Moment

  The maximum moment is at BB′. Factored moment at BB′ is

  $$M_B = 1.50(11.42)\cos(21.14°)(\frac{1}{3})(22.653) = 120.6 \text{ kip-ft}$$

  The factored moment along the stem at the strength limit state is shown in Figure 11.2.2-10.

- Determine Required Factored Flexural Resistance for Minimum Reinforcement Requirement (AASHTO 5.6.3.3)

  Assume the section is noncompression-controlled. From AASHTO 5.4.2.6,

  $$f_r = 0.24\sqrt{f'_c} = 0.24\sqrt{4.0} = 0.48 \text{ ksi}$$

Section modulus,

$$S_c = \frac{1}{6}(12)(28.5^2) = 1624.5 \text{ in.}^3$$

Cracking moment,

$$M_{cr} = \gamma_3 \left[y_1 f_r S_c - M_{dnc} \left(\frac{S_c}{S_{nc}} - 1 \right) \right] \qquad \text{(AASHTO 5.6.3.3-1)}$$

$$S_c = S_{nc}$$

$$\gamma_1 = 1.6$$

$$\gamma_3 = 0.75 \text{ for A706, } f_y = 60 \text{ ksi}$$

$$M_{cr} = 1.2 f_r S_c = 1.2(0.48)(1624.5)\left(\frac{1}{12}\right) = 78.0 \text{ kip-ft}$$

$$1.33 M_u = 1.33(120.6) = 160.4 \text{ kip-ft}$$

The required factored flexural resistance for minimum reinforcement at BB′ is

$$M_{r-\min} = \text{lesser of} \begin{cases} 1.33 M_u \\ M_{cr} \end{cases} = 78.0 \text{ kip-ft}$$

The factored flexural resistance along the stem for minimum reinforcement requirement is shown in Figure 11.2.2-10.

- Calculate Shear Force

The maximum factored shear force for the stem is at the base of the stem because there is no reliable compression introduced by the reaction from the footing.

$$V_B = 1.50(11.42)\cos(21.14°) = 16.0 \text{ kip}$$

The factored shear force along the stem at the strength limit state is shown in Figure 11.2.2-11.

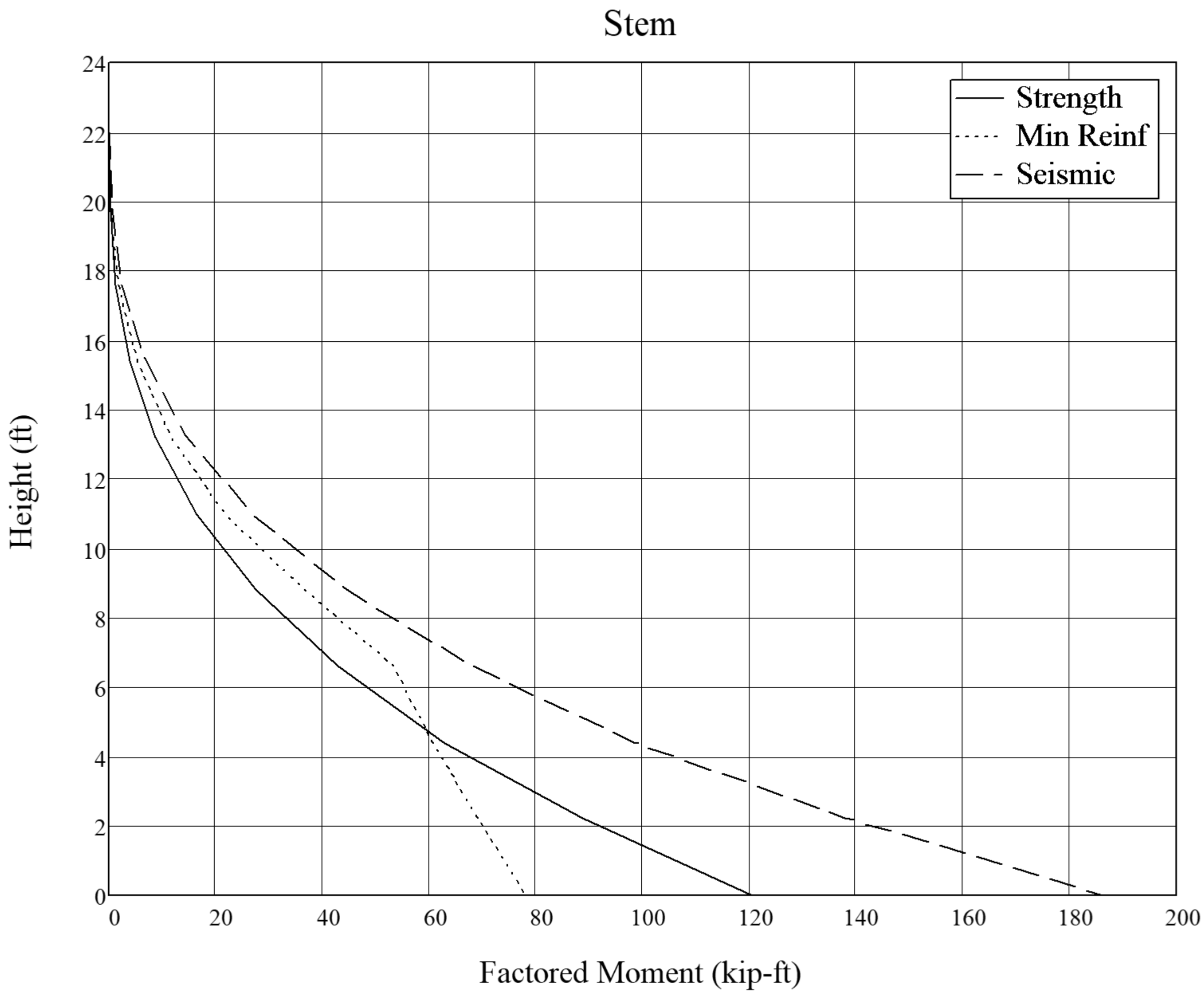


Figure 11.2.2-10 Factored Moment in the Stem

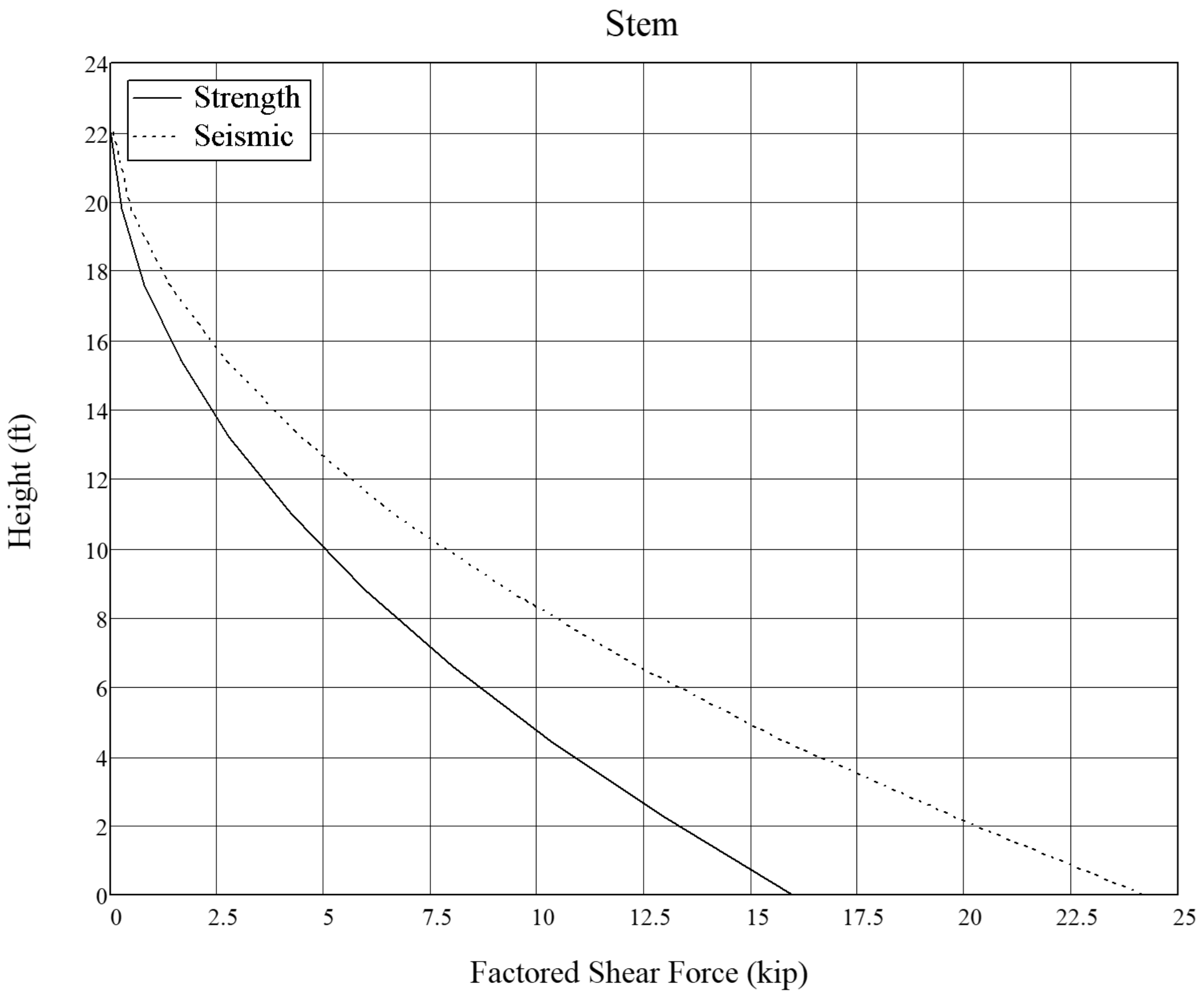


Figure 11.2.2-11 Factored Shear Force in the Stem

<u>Service Limit State</u>

Load factor,

$\gamma_p = 1.0$ for horizontal active earth pressure

- Calculate Moment

  The maximum moment is at BB′. Factored moment at BB′ is

$$M_B = 1.0(11.42)\cos(21.14°)\left(\frac{1}{3}\right)(22.653) = 80.4 \text{ kip-ft}$$

Extreme Limit State

Load factor,

$\gamma_p = 1.0$ for horizontal active earth pressure and inertial force of stem

The stem is subject to seismic soil load and stem inertial force. Note the inertial force of the soil over the heel is not included in the stem design.

The seismic soil load P_{AE} is calculated using the trial wedge method, given $k_h = 0.28$. P_{AE} is assumed to apply at δ_A from the horizontal and to be distributed linearly along AB. A free body diagram is shown in Figure 11.2.2-12. The effect of soil AA′B on the stem is again ignored.

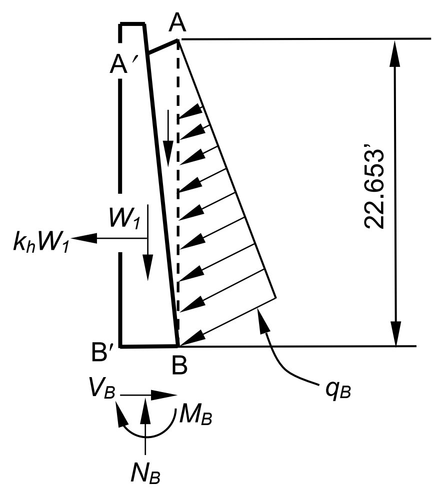


Figure 11.2.2-12 Forces on Stem at Extreme Event (Seismic) Limit State

$$P_{AE} = 24.25 \text{ kip}$$

$$q_B = \frac{2P_{AE}}{H}$$

where $H = 22.653$ ft

$$q_B = \frac{2(24.25)}{22.653} = 2.14 \text{ ksf}$$

The inertia k_hW_1 applies at the stem's center of gravity.

- Calculate Moment

  The factored moment at BB′ due to P_{AE} for a 1-foot wide stem is

$$M_{B_1} = 1.0\left[24.25\cos(21.14°)\left(\frac{1}{3}\right)(22.653)\right] = 170.79 \text{ kip-ft}$$

The factored moment at BB′ due to inertial force on the stem.

$$M_{B_2} = 1.0\left[0.28(0.15)(23)(0.9375)\left(\frac{1}{2}\right)(23) + 0.28(0.15)\left(\frac{1}{2}\right)(23)(1.4375)\left(\frac{1}{3}\right)(23)\right]$$
$$= 1.0[10.415 + 5.323] = 15.74 \text{ kip-ft}$$

The total factored moment at BB′,

$$M_B = 170.79 + 15.74 = 186.5 \text{ kip-ft}$$

The factored moment along the stem at strength limit state is shown in Figure 11.2.2-10.

- Calculate Shear Force

The factored shear force at BB′ due to P_{AE} for a 1-foot wide stem is

$$V_{B_1} = 1.0\left[24.25\cos(21.14°)\right] = 22.62 \text{ kip}$$

The factored shear force at *BB’* due to inertial force on the stem,

$$V_{B_2} = 1.0\left[0.28(0.15)\left(\frac{1}{2}\right)(0.9375 + 2.375)(23)\right] = 1.60 \text{ kip}$$

The total factored shear at BB′,

$$V_B = 22.62 + 1.60 = 24.2 \text{ kip}$$

The factored shear force along the stem at extreme event (seismic) limit state is shown in Figure 11.2.2-11.

Flexure Design

1. Determine the amount of reinforcement at the location with a maximum moment

The section with the maximum moment is at the base of the stem, BB′.

Try #9@8

For a 1-foot wide stem,

$$A_s = 1.0\left(\frac{12}{8}\right) = 1.50 \text{ in.}^2$$

$$c = \frac{A_s f_y}{0.85 f'_c \beta_1 b} \qquad \text{(AASHTO 5.6.3.1.1-4)}$$

$$a = \beta_1 c$$

$$\beta_1 = 0.85$$

$$b = 12 \text{ in.}$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{1.50(60)}{0.85(4.0)(12)} = 2.20 \text{ in.}$$

$$c = \frac{2.20}{0.85} = 2.59 \text{ in.}$$

From extreme compression fiber to the centroid of reinforcement,

$$d_e = 28.5 - 2 - \frac{1}{2}(1.128) = 25.94 \text{ in.}$$

$$M_n = A_s f_y (d_e - \frac{1}{2}a) = 1.50(60)\left[25.94 - \frac{1}{2}(2.20)\right]\left(\frac{1}{12}\right) = 186.3 \text{ kip-ft}$$

Strength Limit State

- Check Tensile Strain in the Rebar (CA 5.6.2.1)

$$\varepsilon_s = 0.003\left(\frac{d_e}{c} - 1\right) = 0.003\left(\frac{25.94}{2.59} - 1\right) = 0.027 > 0.004 \text{ OK}$$

- Check the Flexure Resistance Factor (AASHTO 5.5.4.2) and Assumption of Noncompression-Controlled Section (AASHTO 5.6.3.3)

  Since the calculated tensile strain in the rebar is larger than 0.005, the section is considered as tension controlled. The assumption of the flexural resistance factor $\phi = 0.9$ is valid, and the use of the minimum factored flexural resistance in Article 5.6.3.3 is appropriate.

$$M_r = \phi M_n = 0.9(186.3) = 167.7 > 120.6 \text{ kip-ft OK}$$

Service Limit State

- Check Crack Control (AASHTO 5.6.7)

$$s = \frac{700\gamma_e}{B_s f_{ss}} - 2d_c$$

where:

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

$\gamma_e = 1.0$ for Class I exposure condition

$$d_c = 2 + \frac{1}{2}(1.128) = 2.56 \text{ in.}$$

$$\beta_s = 1 + \frac{2.56}{0.7(28.5 - 2.56)} = 1.14$$

As calculated previously, the moment at section B is,

$M_B = 80.4$ kip-ft

Cracking moment,

$$M_{cr} = f_r S_c = 0.48(1624.5)\left(\frac{1}{12}\right) = 65.0 \text{ kip-ft}$$

$M_B > M_{cr}$

Thus, the section is cracked. Stress in tension steel, f_{ss}, in a cracked section may be calculated with the following procedure.

Modulus of elasticity ratio of steel and concrete,

$$n = \frac{E_s}{E_c} = \frac{29000}{57\sqrt{4000}} = 8$$

$$B = \frac{1}{b}(nA_s) = \frac{1}{12}(8)(1.50) = 1.0$$

where b is the unit width of the section in inches and A_s is the steel area per foot (in^2/ft).

$$C = \frac{2}{b}(ndA_s) = \frac{2}{12}(8)(25.94)(1.50) = 51.88$$

where d is the distance from the tension steel to the extreme compression fiber of the section.

$$x = \sqrt{B^2 + C} - B = \sqrt{1^2 + 51.88} - 1.0 = 6.27$$

Section moment of inertia,

$$I = \frac{1}{3}bx^3 + nA_s(d - x)^2 = \frac{1}{3}(12)(6.27^3) + 8(1.50)(25.94 - 6.27)^2 = 5628.9 \text{ in.}^4$$

Stress in tension steel,

$$f_{ss} = \frac{nM(d-x)}{I} = \frac{8(80.4)(12)(25.94-6.27)}{5628.9} = 27.0 \text{ ksi} < 0.60f_y = 36 \text{ ksi}$$

Maximum allowable rebar spacing,

$$s = \frac{700(1.0)}{1.14(27.0)} - 2(2.56) = 17.6 \text{ in.} > 8 \text{ in.(rebar span at BB')} \text{ OK}$$

Extreme Event (Seismic) Limit State

$$\phi = 1.0$$

$$M_r = \phi M_n = 1.0(186.3) = 186.3 \cong M_u = M_B = 186.5 \text{ kip-ft OK}$$

2. Trim longitudinal bars

A moment in the stem reduces as the distance increases from section BB′. At a certain point on the stem where the moment is approximately half of that at section BB′, half the rebars at section BB′ can be cut off. The moment diagram indicates an extreme event limit state governs the design. Moment for seismic loads consists of inertia induced moment and dynamic horizontal active earth load P_{AE}. Let z be the location of the section measured from the top of the backfill at point "*A*". Thus, the moment induced by the inertia is approximately in proportion to z^2, while the moment induced by P_{AE} is in proportion to z^3. The former is only 10% the latter. Therefore, the moment is approximately in proportion to z^3. The following equation represents the relation when the moment at the location ηz is half the magnitude at z. η is a number less than 1.0:

$$(\eta z)^3 = \frac{1}{2} z^3$$

$\eta = 0.794$, or the rebar may be cut off at about $(1-0.794) = 20\%$ height of the stem from the bottom.

Per AASHTO 5.10.8.1.2a, extend the bar by the greater of

$$\begin{cases} d_v = 25.94 - \frac{1}{2}(2.20) = 24.8 \text{ in.} \\ 15d_b = 16.9 \text{ in.} \end{cases}$$, say 25 in. beyond the theoretical cut off point.

Terminate the trimmed bar at 6′ – 6″ from the bottom of the stem.

Shear Design (AASHTO 5.7.3.3)

The shear design per AASHTO 5.7.3.3 is similar to the heel design. The design process is omitted.

Check Longitudinal Reinforcement Requirement (AASHTO 5.7.3.5)

This requirement is not applicable at the base of the stem, where the section is designed for maximum moment. The section where the longitudinal rebar is cut off needs to be checked for this requirement.

The location to the base of the stem, *h*,

$$h = 6.5 - \frac{30}{12} = 4.0 \text{ ft}$$

In the equation above, *h* refers to the location of the actual cut off minus the rebar development length, 30 in., calculated per CA 5.10.8.2.1d. Only the extreme event limit state (seismic) needs to be checked. At *h* from the bottom of the stem,

$$M_u = 104 \text{ kip-ft}$$

$$V_u = 16.4 \text{ kip}$$

The thickness of the section,

$$t = 2.1 \text{ ft} = 25.2 \text{ in.}$$

$$d_v = 21.4 \text{ in.}$$

$$\theta = 53.4^\circ$$

$$A_s f_y = (1.0)\frac{12}{16}(60) = 45 \text{ kip}$$

$$\frac{|M_u|}{d_v \phi_f} + \frac{|V_u|}{\phi_v}\cot(\theta) = \frac{104(12)}{21.4(1.0)} + \frac{16.4}{1.0}\cot(53.4^\circ) = 70.5 \text{ kip} > A_s f_y \quad \text{N.G.}$$

Extend the bar to be trimmed at 10’-4” from the base of the stem. At this location,

$$d_v = 18.4 \text{ in.}$$

$$\theta = 53.4^\circ$$

$$M_u = 44.3 \text{ kip-ft}$$

$$V_u = 9.3 \text{ kip}$$

$$\frac{|M_u|}{d_v \phi_f} + \frac{|V_u|}{\phi_v}\cot(\theta) = \frac{44.3(12)}{18.4(1.0)} + \frac{9.3}{1.0}\cot(53.4^\circ) = 36.0 \text{ kip} < A_s f_y \quad \text{OK}$$

Check Interface Shear Resistance (AASHTO 5.7.4.3)

The construction joint is at the base of the stem. Shear friction capacity should be checked.

Since the concrete at the construction joint is not intentionally roughened, the following cohesion and friction factors specified in AASHTO Article 5.7.4.4 are used:

Cohesion: $c = 0.075$ ksi

Friction: $\mu = 0.6$

Concrete strength: $K_1 = 0.2$

Maximum friction capacity: $K_2 = 0.8$ ksi

$$V_{ni} = cA_{cv} + \mu(A_{vf} f_y + P_c)$$

$$V_{ni} \le K_1 f'_c A_{cv}$$

Or

$$V_{ni} \le K_2 A_{cv}$$

$$V_{ri} = \phi_v V_{ni}$$

Minimum required steel for a 1-foot wide stem,

$$A_{vf} = 0.05 \frac{A_{cv}}{f_y}$$

$$A_{cv} = 28.5(12) = 342.0 \text{ in.}^2$$

$$A_{vf} = 1.50 \text{ in.}^2$$

Ignore the axial load on stem (conservative assumption)

$$V_{ni} = 0.075(342.0) + 0.6\left[1.50(60) + 0\right] = 79.7 \text{ kip}$$

<u>Strength Limit Sate</u>

$$V_u = 16.0 \text{ kip}$$

$$V_{ri} = \phi V_{ni} = 0.9(79.7) = 71.7 \text{ kip} > V_u \text{ OK}$$

<u>Extreme Event (Seismic) Limit State</u>

$$V_u = 24.2 \text{ kip}$$

$$V_{ri} = \phi V_{ni} = 1.0(79.7) = 79.7 \text{ kip} > V_u \text{ OK}$$

11.2.2.4.2 Toe

Calculate Internal Forces

Strength Limit State

Load factors correspond to those for Strength Limit State Ia,

$\gamma_p = 1.35$ for earth self-weight

$\gamma_p = 1.25$ for concrete self-weight

$\gamma_p = 1.5$ for horizontal active earth pressure

Resistance factors,

$\varphi_{ep} = 0.5$ for horizontal passive earth resistance (CA Table 11.5.7-1)

- Calculate Pressures on the Footing

  Figure 11.2.2-13 shows the applied vertical reaction force and moment about the centerline of the footing, as well as the bearing stress.

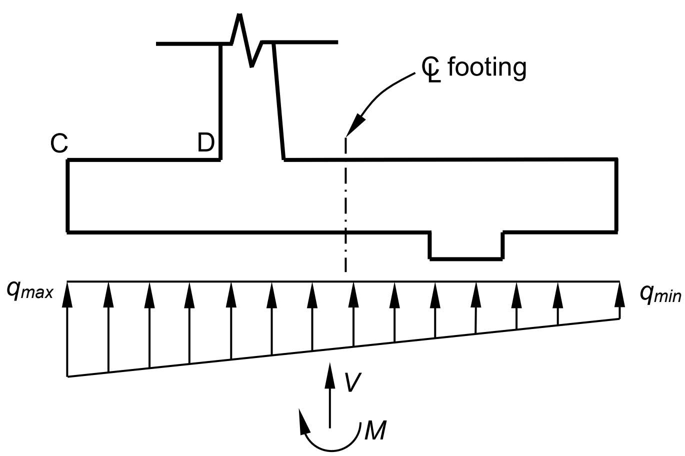


Figure 11.2.2-13 Bearing Stress under the Footing

The eccentricity of the reaction force, *V*, at the bottom of the footing,

$$e = \frac{19}{2} - 8.50 = 1.0 \text{ ft} < \frac{B}{6} = 3.2 \text{ ft}$$

Thus, the footing bearing stress is trapezoidal distributed.

Vertical factored reaction at the bottom of the footing, V, (see Table 11.2.2-4)

$$V = 70.68 \text{ kip}$$

Moment about the center line of the footing, M,

$$M = 70.68(1.0) = 70.68 \text{ kip-ft}$$

The maximum and minimum bearing stress under the footing are,

$$q_{\min}^{\max} = \frac{V}{B}(1 \pm 6\frac{e}{B}) = \frac{70.68}{19}\left[1 \pm (6)\frac{1.0}{19}\right] = \begin{matrix} 4.895 \\ 2.545 \end{matrix} \text{ ksf}$$

Figure 11.2.2-14 shows a free body diagram of the toe. Section E is d_v from the face of the stem and is the critical section for shear design.

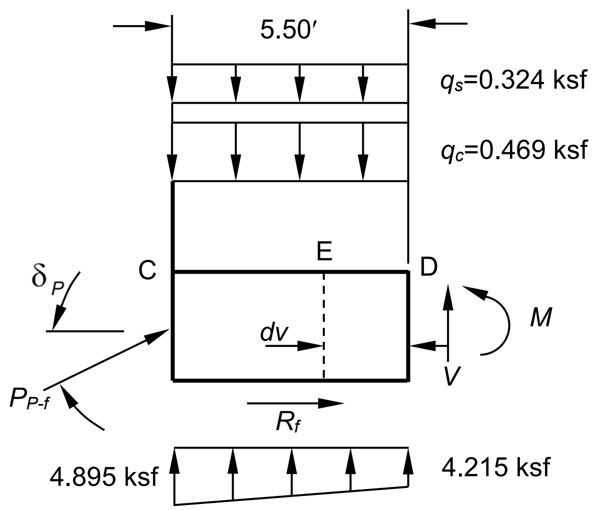


Figure 11.2.2-14 Forces on Toe

Applied forces on the toe are soil passive resistance on the toe, $P_{P\text{-}f}$, overburden soil self-weight, q_s, concrete self-weight, q_c, footing bearing pressure, q_u. Friction at the footing bottom, R_f, is ignored, but this is conservative.

$$q_D = 4.895 - \frac{4.895 - 2.545}{19} 5.5 = 4.215 \text{ksf}$$

Factored overburden soil pressure,

$$q_s = 1.35(0.12)(2) = 0.324 \text{ ksf}$$

Factored concrete self-weight,

$$q_c = 1.25(0.15)(2.5) = 0.469 \text{ ksf}$$

Factored horizontal passive earth resistance on the toe,

$$P_{P-f} = 0.50\left[\frac{1}{2}(7.05)(0.12)(4.5^2 - 2.0^2)\right] = 3.44 \text{ kip}$$

In the above equation, 7.05 is k_p, as calculated before.

- Calculate Moment

The factored moment at point "D" is calculated by summing all the moments from the forces and pressures on the toe about "D",

$$M_D = 65.9 \text{ kip-ft}$$

The factored moment along the toe at the strength limit state is shown in Figure 11.2.2-17.

- Determine Required Factored Flexural Resistance for Minimum Reinforcement Requirement (AASHTO 5.6.3.3)

Assume the section is noncompression-controlled.

$$f_r = 0.48 \text{ ksi}$$

$$S_c = \frac{1}{6}(12)(30^2) = 1800 \text{ in.}^3$$

Cracking moment,

$$M_{cr} = 1.2 f_r S_c = 1.2(0.48)(1800)\frac{1}{12} = 86.4 \text{ kip-ft}$$

$$1.33 M_u = 1.33(65.9) = 87.6 \text{ kip-ft}$$

The minimum factored flexural resistance to be designed for at "D" is

$$M_{r,\min} = 86.4 \text{ kip-ft}$$

The required factored flexural resistance for minimum reinforcement along the toe is shown in Figure 11.2.2-17.

- Calculate Shear Force

The critical section for shear is at d_v from the face of the stem, where the shear force is at the maximum. The critical section is at "E" in Figure 11.2.2-14. According to is at the maximum. The critical section is at “E” in Figure 11.2.2-14. According to AASHTO 5.7.2.8, the effective shear depth is,

$$d_v = 26.44 - \frac{1}{2}(2.20) = 25.34 \text{ in.} > \begin{cases} 0.9d_e = 23.8 \text{ in.} \\ 0.72h = 21.6 \text{ in.} \end{cases}$$

$$V_E = 15.1 \text{ kip}$$

The factored shear force on the toe at the strength limit state is shown in Figure 11.2.2-18.

Extreme Event (Seismic) Limit State

Load factors and resistance factors for the forces on the toe are all equal to 1.0.

- Calculate Pressures on the Footing

  Eccentricity,

$$e = 5.98 \text{ ft} > \frac{B}{6} = 3.2 \text{ ft}$$

Thus, the pressure below the footing is triangularly distributed (see Figure 11.2.2-15). The vertical reaction at the footing bottom,

$$V = 55.36 \text{ kip}$$

Moment about the centerline of the footing,

$$M = 55.36(5.98) = 331.1 \text{ kip-ft}$$

Since the bearing stress under the footing is triangularly shaped, the maximum bearing stress is,

$$q_{\max} = \frac{2V}{3X} = \frac{2(55.36)}{3(3.52)} = 10.48 \text{ ksf}$$

$$L = 3X = 3(3.52) = 10.56 \text{ ft}$$

Figure 11.2.2-15 shows the forces on the footing.

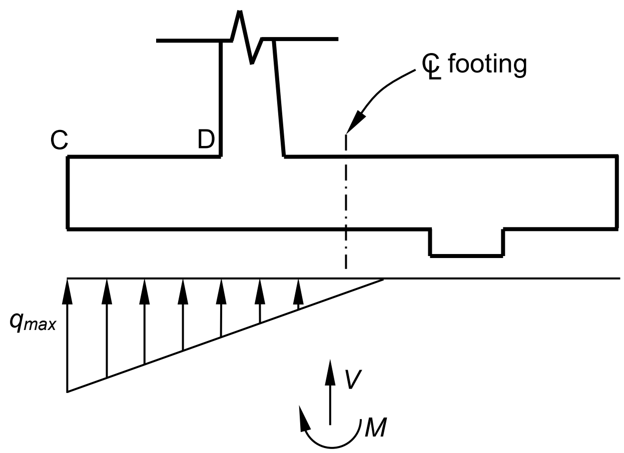


Figure 11.2.2-15 Forces on Footing Bottom for Seismic Limit State

Figure 11.2.2-16 shows forces on the toe. All notations are similar to the strength limit state, except horizontal passive earth resistance on the footing for the seismic limit state, shown as $P_{PE\text{-}f}$.

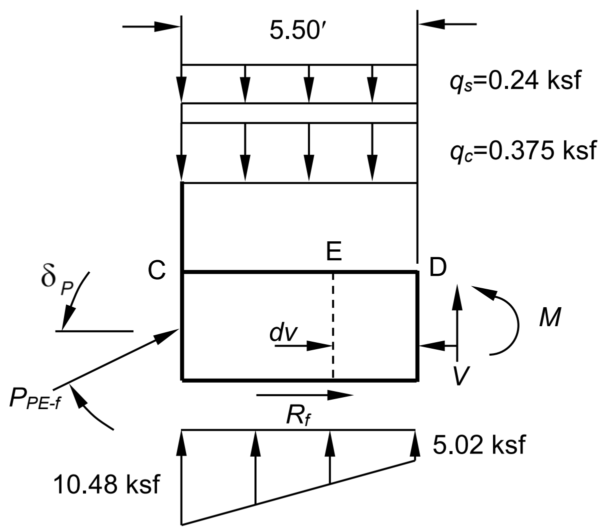


Figure 11.2.2-16 Forces on Toe

Horizontal passive earth resistance on the toe,

$$P_{PE-f} = \frac{1}{2}(5.64)(0.12)(4.5^2 - 2.0^2) = 5.50 \text{ kip}$$

- Calculate Moment

The factored moment at section "D",

$M_D = 133.5$ kip-ft

Minimum design moment

$M_{u-\min} = M_{cr} = 86.4$ kip-ft $< M_D$

Thus, the design moment at section "D",

$M_D = 133.5$ kip-ft

The factored moment along the toe at the extreme event (seismic) limit state is shown in Figure 11.2.2-17.

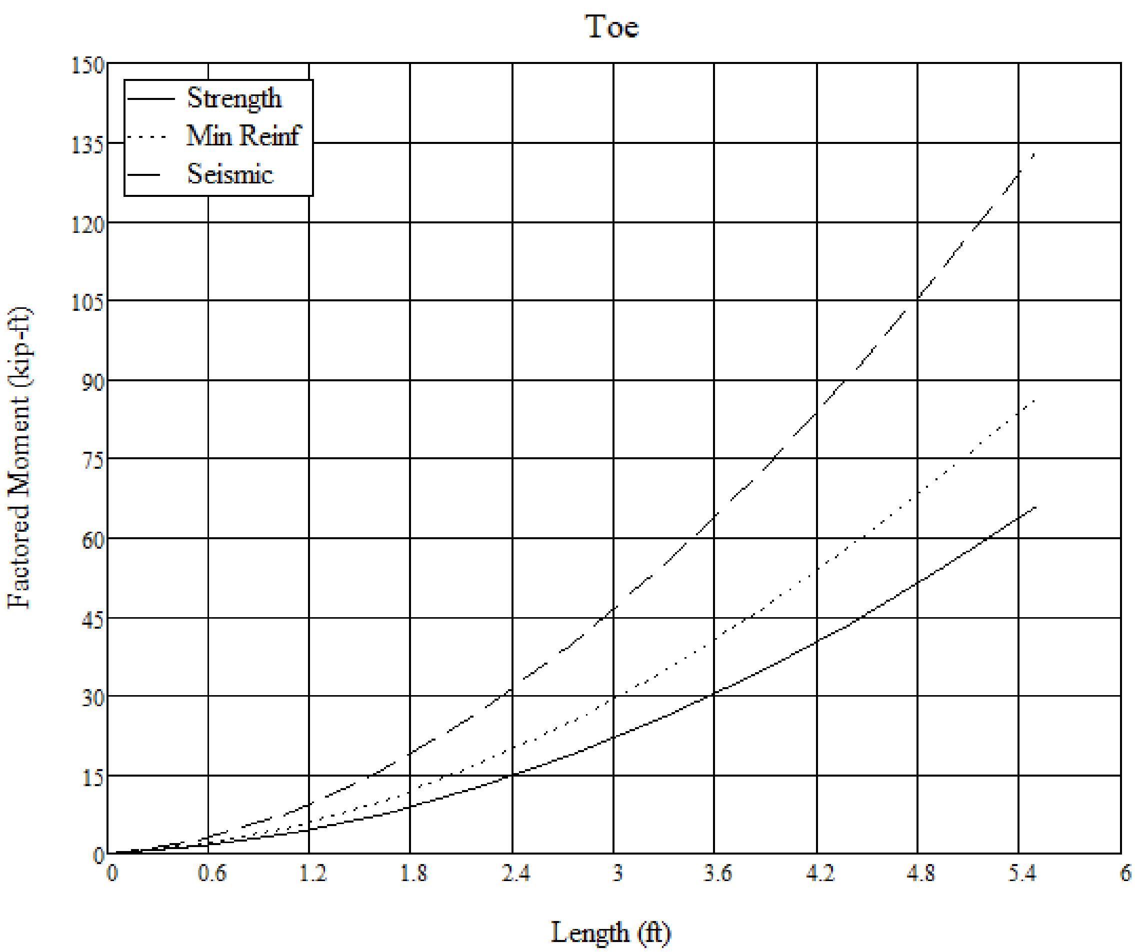


Figure 11.2.2-17 Factored Moment on the Toe

- Calculate Shear Force

  As for the strength limit state, the critical section for shear force for the toe is at “E” in Figure 11.2.2-14. The factored shear force is

  $V_u = 29.8$ kip

  The factored shear force on the toe at the extreme event (seismic) limit state is shown in Figure 11.2.2-18.

Flexure Design

The flexure design of the toe is similar to that of the stem. The design process is omitted.

Shear Design (AASHTO 5.7.3.3)

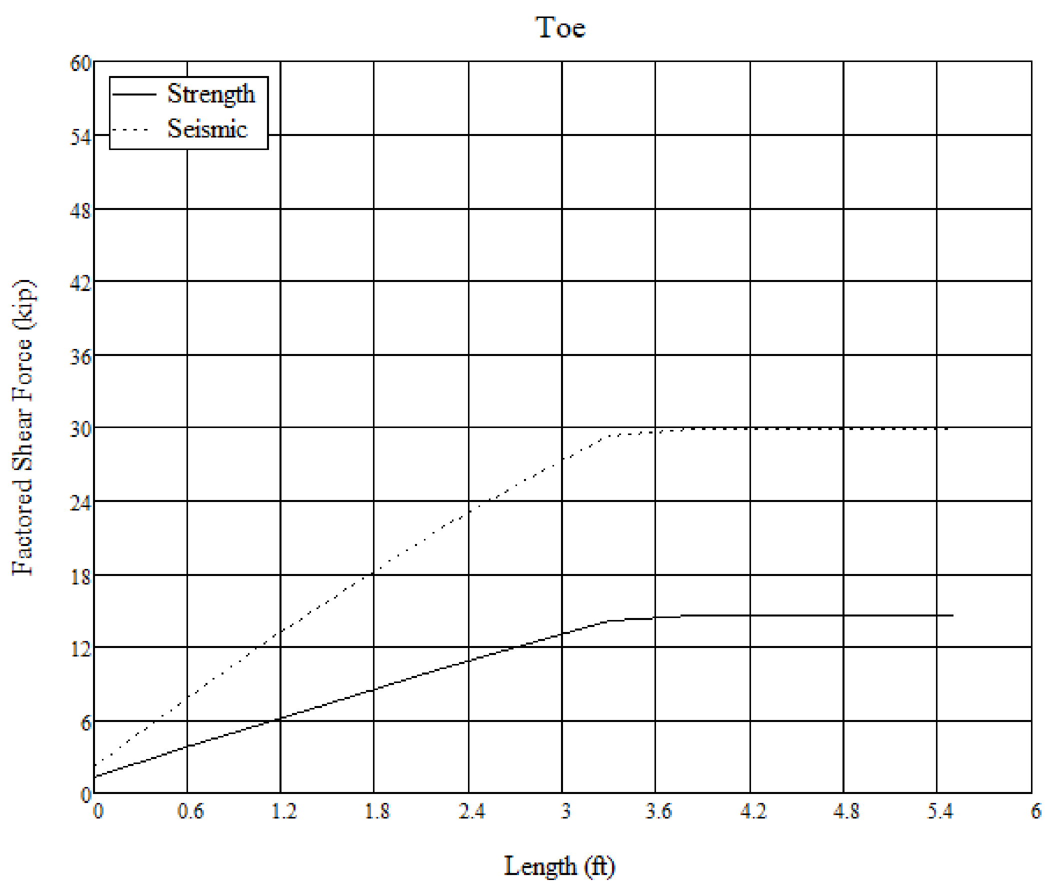


Figure 11.2.2-18 Factored Shear Force on the Toe

The shear design of the toe is similar to that of the stem. The design process is not illustrated.

Check Longitudinal Reinforcement Requirement (AASHTO 5.7.3.5)

No rebar is cut off, so this requirement need not to be checked.

11.2.2.4.3 Heel

The free body diagram of the heel and the soil over the heel is shown in Figure 11.2.2-19a). In the figure, Point "B" is the intersection of the heel and the stem. E is the end of the heel. AB is the vertical plane passing through Point "B". FE is the vertical plane passing through Point "E". AF is the backfill surface. The encircled numbers in the figure are the backfill broken down to simple geometry. Earth pressure FEDD′E′ is what has been used in the external stability analysis. Earth pressure ABB' is the earth pressure applied on the stem. The pressure q_5 and q_6 applied to CD is the bearing stress at the bottom of the footing and is obtained in external stability analysis. Depending on the eccentricity of the reaction at the footing bottom, the bearing stress along CD may be either trapezoidal or triangularly distributed. W_s is the self-weight of the soil over the heel, broken down to parts 1 and 2 in the figure. W_c is the self-weight of the heel and the shear key.

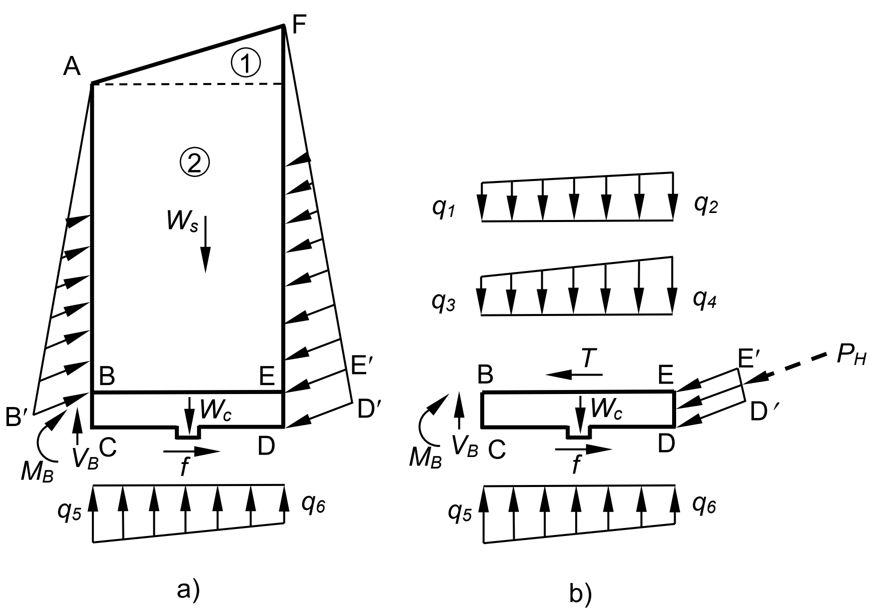


Figure 11.2.2-19 a) Free Body Diagram of Heel and Soil Over Heel; b) Free Body Diagram of Heel

$$\overline{AB} = 22.653 \text{ ft}$$

$$\overline{EF} = 27.94 \text{ ft}$$

$$\overline{BE} = 11.125 \text{ ft}$$

Figure 11.2.2-19b shows the free body diagram of the heel. The pressure to the heel's top consists of soil AFEB (q_1 & q_2) self-weight and the vertical component of the active earth stress at the top of the footing (q_3 & q_4).

For the example problem, it is obvious that the factored pressure by the self-weight of ABEF is,

$$q_1 = \gamma_p \gamma_s \overline{AB}$$

$$q_2 = \gamma_p \gamma_s \overline{EF}$$

The factored pressure on top of the footing due to active soil pressure at Point "B" and Point "E" is

$$q_3 = \gamma_p \gamma_s k_{AB} \overline{AB} \tan(\beta_{AB}) \sin(\beta_{AB})$$

$$q_4 = \gamma_p \gamma_s k_{EF} \overline{EF} \tan(\beta_{EF}) \sin(\beta_{EF})$$

where:

$k_{AB} = k_a$ at AB for static load

$k_{AB} = k_{ae}$ at AB for seismic load

$k_{EF} = k_a$ at EF for static load

$k_{EF} = k_{ae}$ at EF for seismic load

The shear force, T, on top of the heel in Figure 11.2.2-19b is the sum of the shear stress from the active pressure over the length of the heel. The shear force is ignored in designing the heel because of the small moment arms.

The resultant of the pressure EDD′E′, P_H, in Figure 11.2.2-19b is assumed to be applied at the mid-point of ED. The error from this assumption is minor because of the small distance between the location of P_H and the mid-point of ED.

Calculate Internal Forces

<u>Strength Limit State Ia</u>

Use maximum load factors with this load combination. The load factors are,

$\gamma_p = 1.35$ for soil self-weight

$\gamma_p = 1.25$ for concrete self-weight

$\gamma_p = 1.50$ for horizontal active earth load

$$q_1 = 1.35(0.12)(22.653) = 3.670 \text{ ksf}$$

$$q_2 = 1.35(0.12)(27.94) = 4.526 \text{ ksf}$$

The unfactored horizontal active earth load on the stem, P_A, and the angle from the horizontal, δ_A, respectively, are

$$P_A = 11.42 \text{ kip}$$

$$\delta_A = 21.14^\circ$$

Since

$$P_A = \gamma_p \frac{1}{2} k_{AB} \gamma_s \overline{\text{AB}}^2$$

$$k_{AB}\gamma_s \overline{\text{AB}} = \frac{2P_A}{\overline{\text{AB}}}$$

The factored stress on top of the heel due to the horizontal active earth pressure at the stem is,

$$q_3 = \gamma_p k_{AB} \gamma_s \overline{\text{AB}} \tan(\beta_{\text{AB}})\sin(\beta_{\text{AB}}) = \gamma_p \frac{2P_A}{\overline{\text{AB}}} \tan(\beta_{\text{AB}})\sin(\beta_{\text{AB}})$$

$$q_3 = (1.5)\frac{2(11.42)}{22.653}\tan(21.14^\circ)\sin(21.14^\circ) = 0.211 \text{ ksf}$$

The unfactored horizontal active earth load on FED is

$$P_A = 17.54 \text{ kip}$$

The unfactored horizontal active earth load on *FE*, and the angle from the horizontal, δ_A, respectively, are

$$P_{A,FE} = 17.54\left(\frac{\overline{FE}}{FED}\right)^2 = 17.54\left(\frac{27.94}{30.44}\right)^2 = 14.78 \text{ kip}$$

$$\delta_A = 9.74^\circ$$

The factored pressure on top of the heel due to active earth pressure at the end of the heel is,

$$q_4 = \gamma_p k_{\text{EF}} \gamma_s \overline{\text{EF}} \tan(\beta_{\text{FE}})\sin(\beta_{\text{FE}}) = \gamma_p \frac{2P_A}{\overline{\text{EF}}} \tan(\beta_{\text{FE}})\sin(\beta_{\text{FE}})$$

$$q_4 = (1.5)\frac{2(14.78)}{27.94}\tan(9.74^\circ)\sin(9.74^\circ) = 0.046 \text{ ksf}$$

The factored resultant of horizontal active earth pressure EDD′E′,

$$P_H = \gamma_p (17.54)\left(1 - \frac{\overline{FE}^2}{FED^2}\right) = 1.5(17.54)\left(1 - \frac{27.94^2}{30.44^2}\right) = 4.14 \text{ kip}$$

From the external stability analysis, the factored bearing stresses are calculated to be,

$$q_5 = 3.921 \text{ ksf}$$

$$q_6 = 2.543 \text{ ksf}$$

The shear force at the bottom of the heel, *f*, is also ignored for its effect on the moment of the heel.

The factored self-weight of the heel is uniformly distributed load,

$$1.25(0.15)(2.5) = 0.47 \text{ ksf}$$

The factored self-weight of the shear key is,

$$1.25(0.15)(0.75)(2.0) = 0.28 \text{ kip}$$

- Calculate Moment

  The factored moment at "*B*" can be calculated by summing all the moments about "*B*",

  $$M_B = 121.4 \text{ kip-ft}$$

  The factored moment along the heel at Strength Limit State Ia is shown in Figure 11.2.2-20.

- Determine Required Factored Flexural Resistance for Minimum Reinforcement Requirement (AASHTO 5.6.3.3)

  Assume the section is noncompression-controlled. As calculated in the toe design,

  $$M_{cr} = 86.4 \text{ kip-ft}$$

  $$1.33M_u = 1.33(121.4) = 161.5 \text{ kip-ft}$$

  Thus, the required minimum flexural resistance is

  $$M_{B,\min} = 86.4 \text{ kip-ft}$$

The required minimum flexural resistance along the heel is shown in Figure 11.2.2-20.

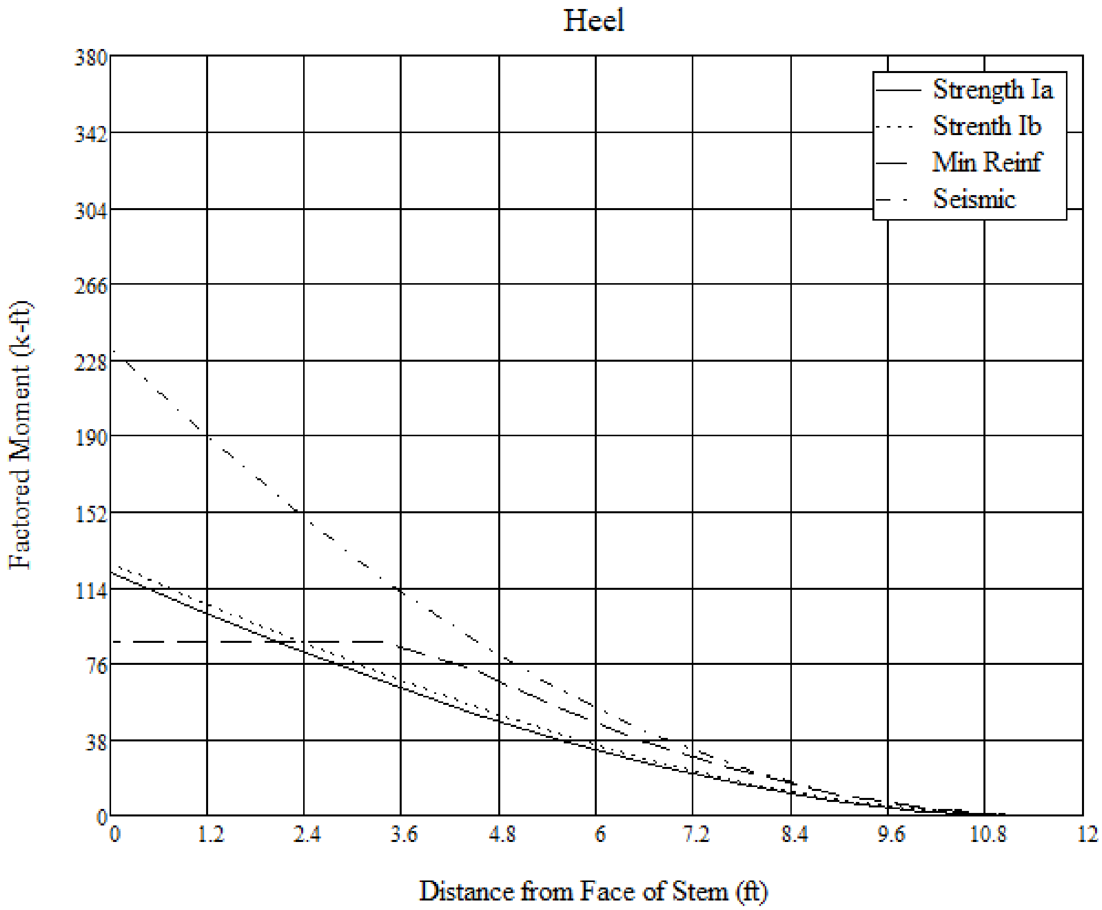


Figure 11.2.2-20 Factored Moment on the Heel

- Calculate Shear Force

  The design shear at “B” is

  $V_B = 17.2$ kip

The factored shear force along the heel at Strength Limit State Ia is shown in Figure 11.2.2-21.

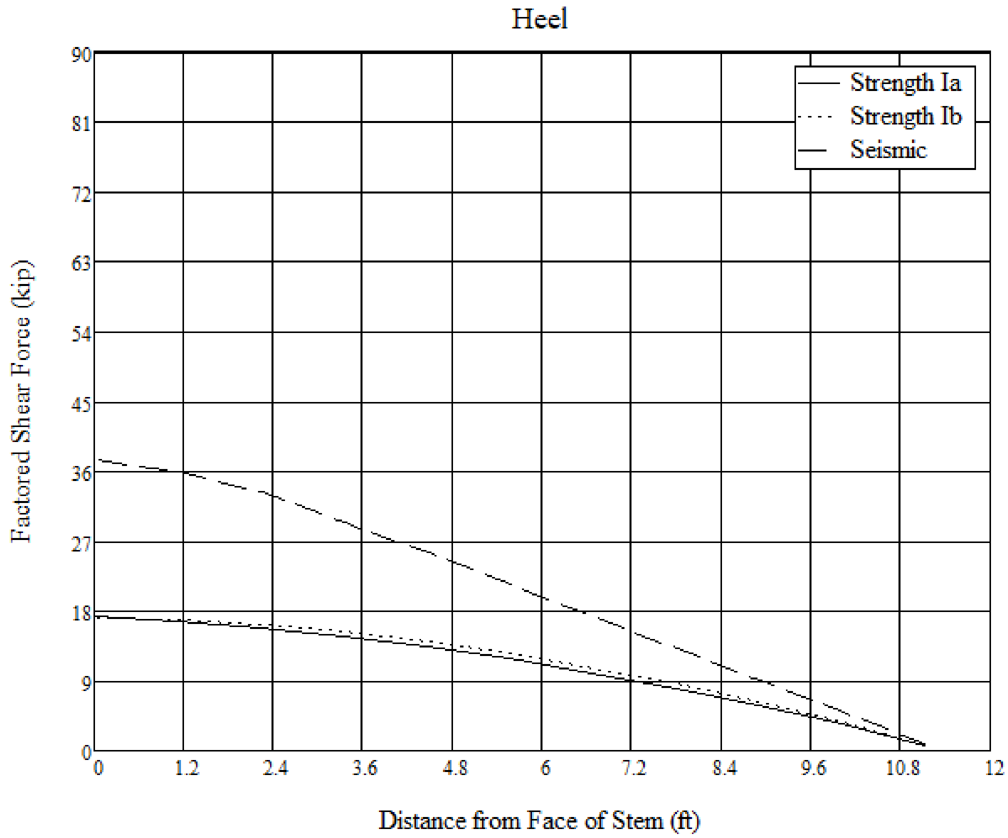


Figure 11.2.2-21 Factored Shear Force on the Heel

Strength Limit State Ib

Use minimum load factors with this load combination. The load factors are

$\gamma_p = 1.00$ for soil self-weight

$\gamma_p = 0.90$ for concrete self-weight

$\gamma_p = 1.50$ for horizontal active earth load

$q_1 = 1.00(0.12)(22.653) = 2.718 \text{ ksf}$

$q_2 = 1.00(0.12)(27.94) = 3.353 \text{ ksf}$

The factored bearing stresses due to horizontal active earth pressure are,

$$q_3 = (1.5)\frac{2(11.42)}{22.653}\tan(21.14°)\sin(21.14°) = 0.211 \text{ ksf}$$

$$q_4 = (1.5)\frac{2(14.78)}{27.94}\tan(9.74°)\sin(9.74°) = 0.0461\text{ ksf}$$

The factored resultant of horizontal active earth pressure EDD′E′ is the same as in Strength Limit Ia,

$$P_H = 4.14\text{ kip}$$

From external stability analysis, the factored bearing stresses are calculated to be,

$$q_5 = 3.109\text{ ksf}$$

$$q_6 = 0.982\text{ ksf}$$

The factored self-weight of the heel is a uniformly distributed load,

$$0.90(0.15)(2.5) = 0.338\text{ ksf}$$

The factored self-weight of the shear key is,

$$0.90(0.15)(0.75)(2.0) = 0.20\text{ kip}$$

- Calculate Moment

  The factored moment at “B”,

  $$M_B = 125.9\text{ kip-ft}$$

  The factored moment along the heel at Strength Limit State Ib is shown in Figure 11.2.2-20

- Check Minimum Design Moment (AASHTO 5.6.3.3)

  This calculation is the same as for Strength Limit State Ia.

- Calculate Shear Force

  The design shear at “B” is

  $$V_B = 17.1\text{ kip}$$

  The factored shear force along the heel at Strength Limit State Ib is shown in Figure 11.2.2-18.

<u>Extreme Event (Seismic) Limit State</u>

All load factors and resistance factors for the structural design are 1.0.

$$q_1 = 1.00(0.12)(22.653) = 2.718\text{ ksf}$$

$$q_2 = 1.00(0.12)(27.94) = 3.353\text{ ksf}$$

The factored horizontal active earth load on the stem, P_{AE}, and the angle from the horizontal, δ_A, respectively, are:

$$P_{AE} = 24.25 \text{ kip}$$

$$\delta_A = 21.14°$$

Since

$$P_{AE} = \gamma_p \frac{1}{2} k_{\text{AB}} \gamma_s \overline{\text{AB}}^2$$

$$k_{AB} \gamma_s \overline{\text{AB}} = \frac{2P_{AE}}{\overline{\text{AB}}}$$

$$q_3 = \gamma_p k_{\text{AB}} \gamma_s \overline{\text{AB}} \tan(\beta_{\text{AB}}) \sin(\beta_{\text{AB}}) = \gamma_p \frac{2P_{AE}}{\overline{\text{AB}}} \tan(\beta_{\text{AB}}) \sin(\beta_{\text{AB}})$$

$$q_3 = (1.00) \frac{2(24.25)}{22.653} \tan(21.14°) \sin(21.14°) = 0.299 \text{ ksf}$$

The unfactored horizontal active earth load on FED is:

$$P_{AE} = 31.56 \text{ kip}$$

The unfactored horizontal active earth load on FE, and the angle from the horizontal, δ_A, respectively, are

$$P_{AE,\text{FE}} = 31.56 \left(\frac{\overline{FE}}{FED} \right)^2 = 31.56 \left(\frac{27.94}{30.44} \right)^2 = 26.59 \text{ kip}$$

$$\delta_A = 9.74°$$

$$q_4 = \gamma_p k_{\text{EF}} \gamma_s \overline{\text{EF}} \tan(\beta_{\text{FE}}) \sin(\beta_{\text{FE}}) = \gamma_p \frac{2P_{AE,\text{FE}}}{\overline{\text{EF}}} \tan(\beta_{\text{FE}}) \sin(\beta_{\text{FE}})$$

$$q_4 = (1.0) \frac{2(26.59)}{27.94} \tan(9.74°) \sin(9.74°) = 0.055 \text{ ksf}$$

The factored resultant of horizontal active earth pressure EDD′E′,

$$P_H = \gamma_p (31.56) \left(1 - \frac{\overline{\text{FE}}^2}{\text{FED}^2} \right) = 1.0(31.56) \left(1 - \frac{27.94^2}{30.44^2} \right) = 4.97 \text{ kip}$$

From the external stability analysis, the bearing stress are triangularly distributed. As shown in Figure 11.2.2-22, the factored bearing stress on the heel at the stem side is

$$q_5 = 2.661 \text{ ksf}$$

The length of the bearing stress under the heel is,

$$L = 2.674 \text{ ft}$$

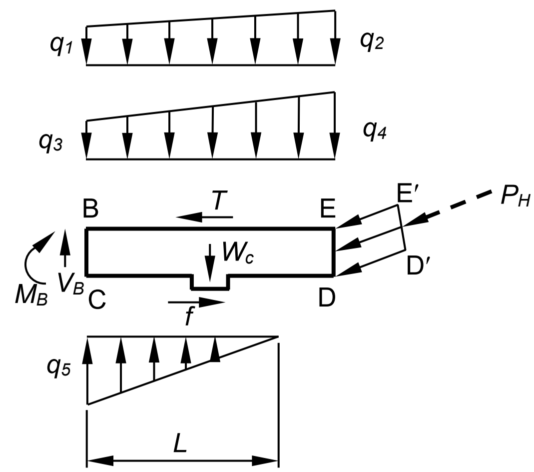


Figure 11.2.2-22 Forces on Heel at Extreme Limit State

The factored self-weight of the heel is uniformly distributed load,

$(1.0)(0.15)(2.5) = 0.375$ ksf

The factored self-weight of the shear key is,

$(1.0)(0.15)(0.75)(2.0) = 0.22$ kip

- Calculate Moment

  The factored moment at "B",

  $M_B = 233.6$ kip-ft

  The factored moment along the heel at extreme event (seismic) limit state is shown in Figure 11.2.2-20.

- Calculate Shear Force

  The design shear at "B" is

  $V_B = 37.4$ kip

  Factored shear force along the heel at extreme event (seismic) limit state is shown in Figure 11.2.2-21.

Flexure Design

The flexure design of the heel is similar to that of the stem. The design process is not illustrated.

Check Longitudinal Reinforcement Requirement (AASHTO 5.7.3.5)
The check process is similar to that for the stem. The check process is omitted.

Shear Design (AASHTO 5.7.3.3)

Point "B" is the critical section at the end of the heel. Extreme event limit state (seismic) governs the shear design. The effective shear depth is determined per AASHTO 5.7.2.8,

d_v is calculated to be 24.2 inches (calculation is not shown here).

$$\because d_v = 24.2 \text{ in.} > \text{greater of} \begin{cases} 0.9d_e = 0.9(30 - 3 - (1.41/2)) = 23.7 \text{ in.} \\ 0.72h = 0.72(30) = 21.6 \text{ in.} \end{cases}$$

$$\therefore d_v = 24.2 \text{ in.}$$

Factored shear and moment at point "*B*",

$$V_B = 37.4 \text{ kip}$$

$$M_B = 233.6 \text{ kip-ft}$$

- Determine θ, β (AASHTO Appendix B5)

Assume $\theta = 45°$

$$M_{u,\min} = V_u d_v = 37.4(24.2)\left(\frac{1}{12}\right) = 75.4 \text{ kip-ft}$$

$$\therefore M_B = 233.6 \text{ kip-ft}$$

$$A_s = 3.0 \text{ in.}^2$$

$$\varepsilon_x = \frac{\frac{233.6(12)}{24.2} + 0.5(37.4)\cot(45°)}{29000(3.0)} = 0.00155$$

$$s_{xe} = (24.2)\frac{1.38}{1.5 + 0.63} = 15.7 \text{ in.} < 80 \text{ in.}$$

$$\therefore s_{xe} = 15.7 \text{ in.}$$

From AASHTO Table B5.2-2 (with no shear reinforcement),

$$\theta = 48.55°$$

$$\beta = 1.57$$

$$\varepsilon_x = \frac{\frac{233.6(12)}{24.2} + 0.5(37.4)\cot(48.55°)}{29000(3.0)} = 0.00152$$

The updated ε_x is close enough and θ and β have converged.

$$V_c = 0.0316\beta\sqrt{f'_c}\, b_v d_v = 0.0316(1.57)\sqrt{4.0}(12)(24.2) = 28.8 \text{ kip}$$

$$V_n = V_c = 28.8 \text{ kip}$$

$$V_r = \phi V_n = 1.0(28.8) = 28.8 \text{ kip} < V_B \text{ N.G.}$$

Thus, transverse reinforcement is necessary.

Try #5 stirrup @16 in both longitudinal direction (along the wall length) and transverse direction (normal to the wall length). The area of transverse reinforcement per foot is,

$$A_v = 0.31\left(\frac{12}{16}\right) = 0.23 \text{ in.}^2$$

Minimum transverse reinforcement (AASHTO 5.7.2.5)

$$A_{v,\min} = 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} = 0.0316\sqrt{4.0}\,\frac{(12)(16)}{60} = 0.20 \text{ in.}^2$$

$$A_v > A_{v,\min} \text{ OK}$$

Maximum spacing of the transverse reinforcement (AASHTO 5.7.2.6)

$$\frac{v_u}{f'_c} = \frac{37.4}{12(24.2)(4.0)} = 0.032 < 0.125$$

$$s_{\max} = 0.8d_v = 0.8(24.2) = 20 \text{ in.}$$

Provided stirrups spacing is 16 in. OK.

- Determine θ, β (AASHTO Appendix B5)

Use AASHTO Eq B5.2-4,

$$M_u = 233.6 \text{ kip-ft}$$

$$V_u = 37.4 \text{ kip}$$

$$A_s = 3.0 \text{ in.}^2$$

$$d_v = 24.2 \text{ in.}$$

Assume $\varepsilon_x = 0.001$

From AASHTO Table B5.2-1 (with transverse reinforcement),

$$\theta = 36.4°$$

$$\beta = 2.23$$

$$\varepsilon_x = \frac{\dfrac{233.6(12)}{24.2} + 0.5(37.4)\cot(36.4°)}{2(29000)(3.0)} = 0.000811$$

Again, from AASHTO Table B5.2-1,

$$\theta = 34.47°$$

$$\beta = 2.34$$

θ, β have converged.

$$V_s = \frac{A_v f_y d_v \cot(\theta)}{s} = \frac{0.23(60)(24.2)\cot(34.47^\circ)}{16} = 30.4 \text{ kip}$$

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v = 0.0316(2.34)\sqrt{4.0}(12)(24.2) = 42.9 \text{ kip}$$

$$V_n = V_c + V_s = 73.3 \text{ kip}$$

$$V_r = \phi V_n = 1.0(73.3) = 73.3 \text{ kip} > V_B \text{ OK}$$

Use #5 stirrups @16 in both directions. It will be enough to provide two rows in the transverse direction.

Temperature and Shrinkage Reinforcement (AASHTO 5.10.6, C5.10.6)

$$A_s = 0.0018bh \text{ (AASHTO C5.10.6)}$$

Stem

a. Along Stem Length –Vertical Rebars

The largest cross section area in the stem is at the bottom. Use this cross section for temperature and shrinkage reinforcement for the whole stem (thickness = 28.5 inches). For a 1-foot length:

$$A_s = 0.0018(28.5)(12) = 0.62 \text{ in.}^2$$

which is 0.31 in.^{2}/ft on each side of the stem. The backfill side has main flexure reinforcement, so this side has enough reinforcement for the temperature and shrinkage requirement.

Provide #6 @ 16 on the toe side of the stem. For a 1-foot length,

$$A_{s,provided} = 0.33 \text{ in.}^2 \text{ OK}$$

b. Horizontal Rebars

Divide height of the stem into two equal segments (the 22-foot design height measured from the bottom of the stem to the point where the backfill intersects the stem). The maximum thickness of the top half segment is

$$t = 20.25 \text{ in.}$$

For a 1-foot length,

$$A_s = 0.0018(20.25)(12) = 0.44 \text{ in.}^2$$

which is 0.22 in^2/ft on each side of the stem. Provide #5 @16 on each side for a 1-foot length,

$$A_{s,provided} = 0.23 \text{ in.}^2 \text{ OK}$$

Provide #5 @16.

The maximum thickness of the lower half segment is

$t = 28.5$ in.

For a 1-foot length,

$$A_s = 0.0018(28.5)(12) = 0.62 \text{ in.}^2$$

which is 0.31 in.^{2}/ft on each side of the stem. Provide #6 @16 on each side for a 1-foot length,

$$A_{s,provided} = 0.33 \text{ in.}^2 \text{ OK}$$

Provide #6 @16.

Footing

In both longitudinal and transverse directions per foot,

$$A_s = 0.0018(30)(12) = 0.65 \text{ in.}^2$$

which is 0.33 in.^{2}/ft on top or bottom surface. Provide #6 @16 on each face in each direction,

$$A_{s,provided} = 0.33 \text{ in.}^2 \text{ OK}$$

Provide #6 @16 in. longitudinal and transverse directions on top and bottom faces of the footing.

11.2.2.5 Final Details

Figure 11.2.2-23 shows the reinforcement and the cross-section details of the retaining wall. The lengths of flexural reinforcement are determined considering the development length requirements in AASHTO 5.10.8.

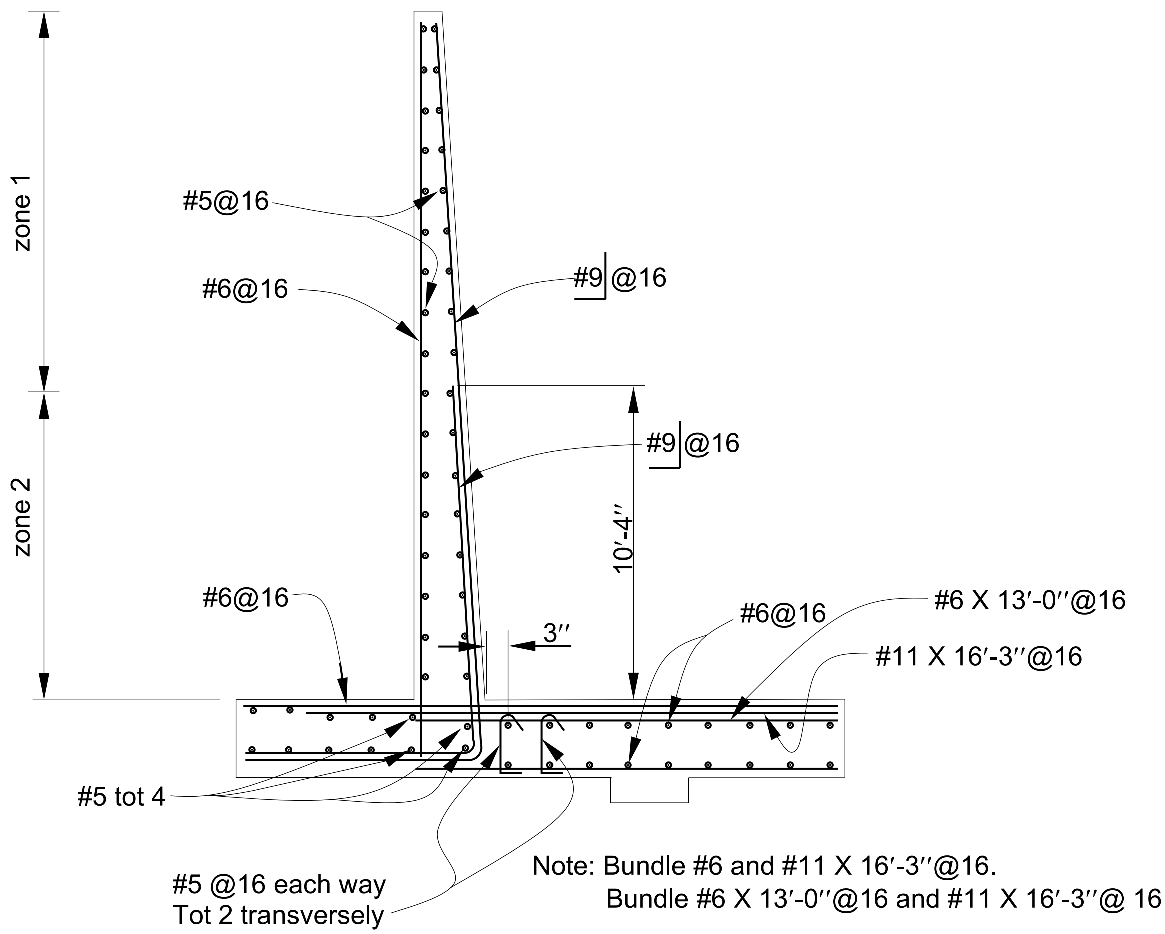


Figure 11.2.2-23 Reinforcement Details

11.2.3 CANTILEVERED SOLDIER PILE WALL DESIGN EXAMPLE

11.2.3.1 Cantilevered Soldier Pile Wall Data

A cantilevered soldier pile lagging wall with piles 8-foot on center, encased in 2-foot diameter drilled holes, filled with concrete backfill below timber lagging (wood lagging), and filled with lean concrete in the area of wood lagging. Soil properties are shown in Figure 11.2.3-1.

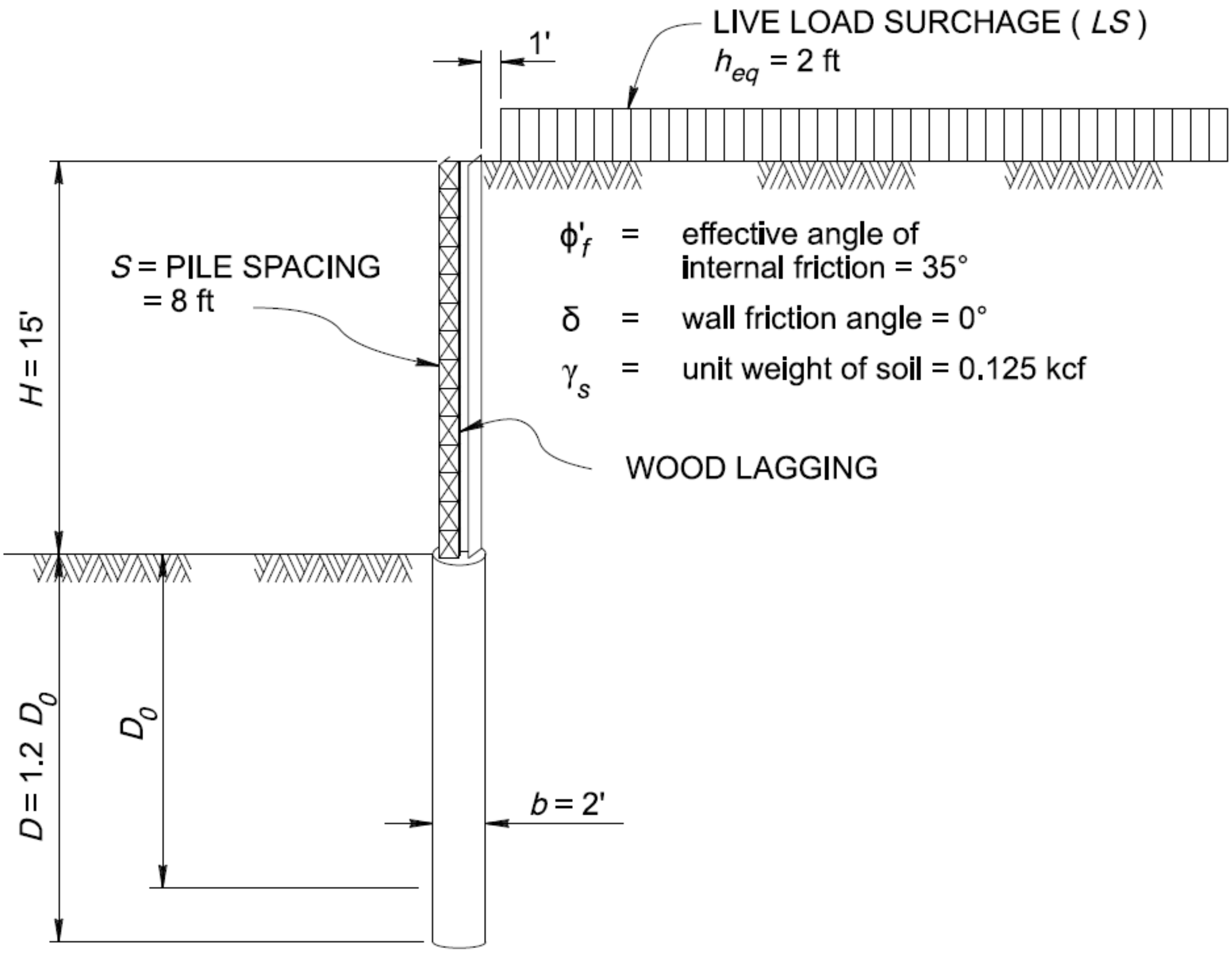


Figure 11.2.3-1 Cantilevered Soldier Pile Wall

Perform the design calculations for the cantilevered soldier pile wall in accordance with the AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019), and the Caltrans Trenching and Shoring Manual (T&S) (Caltrans, 2011) for the following Limit States:

- Service I Limit State
- Strength I Limit State

Table 11.2.3-1 summarizes the load factors for the target limit states:

Table 11.2.3-1 Horizontal Earth Pressure (*EH*) and Live Load Surcharge (*LS*) Load Factors

Limit State	Active *EH*	*LS*	Reference
Service I	1.0	1.0	AASHTO Table 3.4.1-2 and CA Table 3.4.1-1
Strength I	1.5	1.75	

Passive lateral earth pressure is considered a resistance and not a load, with the following resistance factors, ϕ:

Table 11.2.3-2 Passive Lateral Earth Pressure Resistance Factors

Limit State	Passive Lateral Earth Pressure	Reference
Service I	1.0	AASHTO Article 11.5.7
Strength I	1.0	CA Table 11.5.7-1

The following resistance factors, ϕ, are used for wood lagging analysis:

Table 11.2.3-3 Wood Lagging Resistance Factors

Limit State	Flexure	Shear	Compression Perpendicular to Grain	Reference
Service I	N/A	N/A	1.0	AASHTO Article 8.5.2.2
Strength I	0.85	0.75	0.9	

The adjusted design values for wood members used in this example problem are shown below:

Table 11.2.3-4 Wood Lagging Adjusted Design Values

Limit State	Flexure	Shear	Compression Perpendicular to Grain	Reference
Service I	N/A	N/A	0.45 ksi	AASHTO Article 8.4.4.1
Strength I	1.5 ksi	0.14 ksi	0.45 ksi	

Note: In this example, the adjusted design values are based on the maximum allowable stresses listed in the Caltrans Standard Specifications 2022, Section 48-2.02B(3)(b). When further guidance is received from the Earth Retaining Systems Committee on the format conversion factor, C_{kf}, for wood lagging, the designer may calculate the adjusted design values per AASHTO LRFD BDS-08.

The following resistance factors are used for steel pile analysis:

Table 11.2.3-5 Steel Pile Resistance Factors

Limit State	Flexure	Shear	Reference
Strength I	0.9	1.0	CA Table 11.5.7-1 and AASHTO Article 6.5.4.2

11.2.3.2 Service I Limit State Checks

11.2.3.2.1 Movement (Refer to AASHTO Article 11.8.3.1)

The following calculations are needed as intermediate steps in order to check pile and lagging deflections:

- Calculate active and passive earth pressure coefficients

  Since the slope angle in front of the wall and the wall friction angle, δ, are zero in this example, use Rankine's Earth Pressure Theory to calculate both the active and passive earth pressure coefficients.

  $$k_a = \tan^2(45 - \frac{\phi'_f}{2}) \qquad \text{(T\&S 4-9 \& AASHTO C11.10.6.2.1-1)}$$

  $$k_a = \tan^2(45 - \frac{35}{2}) = 0.271$$

  $$k_p = \tan^2(45 + \frac{\phi'_f}{2}) \qquad \text{(T\&S 4-10)}$$

  $$k_p = \tan^2(45 + \frac{35}{2}) = 3.690$$

  Note: Rankine's equation for the passive earth pressure should not be used if the slope angle in front of the wall is greater than zero (Refer to T&S 4.3). Also, Rankine's equation underestimates the passive earth pressure if $\delta > 0$. For passive earth pressure, it is recommended to use Coulomb's Theory (T&S Eq. 4-22) when $0 < \delta \leq 1/3\phi'_f$, and the Log-Spiral Method (AASHTO Figure 3.11.5.4-1) when $\delta > 1/3\phi'_f$. AASHTO Article C3.11.5.3 gives guidance on the determination of δ. In this example, $\delta = 0$ for simplicity.

- Calculate factored lateral earth pressure distributions

  Factored active EH at excavation line:

  $$p_a = k_a \gamma_s H \gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$p_a = 0.271(0.125)(15)(1.0) = 0.508$ ksf

where:

$\gamma_p =$ active EH load factor $= 1.0$ (Refer to Table 11.2.3-1)

Factored active *EH* at depth *Do*:

$p_{aD_0} = k_a \gamma_s (H + D_0)\gamma_p$ (Per AASHTO 3.11.5.1-1)

$p_{aD_0} = 0.271(0.125)(15 + D_0)(1.0) = 0.508 + 0.034D_0$ ksf

Factored passive lateral earth pressure at depth *Do*:

$p_p = k_p \gamma_s D_0 \phi$ (Per AASHTO 3.11.5.1-1)

$p_p = 3.690(0.125)(D_0)(1.0) = 0.461D_0$ ksf

where:

$\phi =$ passive lateral earth pressure resistance factor $= 1.0$ (Refer to Table 11.2.3-2

Factored lateral earth pressure due to *LS* above the excavation line only:

$\Delta_p = k_a \gamma_s h_{eq} \gamma$ (Per AASHTO 3.11.6.4-1)

$\Delta_p = 0.271(0.125)(2)(1.0) = 0.068$ ksf

where:

$h_{eq} =$ equivalent height of soil for vehicular load $= 2$ ft (Refer to AASHTO Table 3.11.6.4-2)

$\gamma =$ LS load factor $= 1.0$ (Refer to Table 11.2.3-1)

- Calculate factored resultant earth forces for a single pile (Refer to AASHTO Article 3.11.5.6)

Factored active earth force above excavation line using 8 feet pile spacing:

$P_{a1} = 0.5 p_a HS = 0.5(0.508)(15)(8) = 30.48$ kip

Factored active earth forces below excavation line using 2 feet diameter hole:

$P_{a2_1} = p_a D_0 b = 0.508D_0(2) = 1.016D_0$ kip

$P_{a2_2} = 0.5(0.034D_0)(D_0)(2) = 0.034D_0^2$ kip

Factored passive earth force below the excavation line using 2 feet diameter hole and soil arching capability factor:

For soldier piles, a soil arching capability factor, f_p, needs to be calculated and applied to the passive forces only (Refer to AASHTO Articles C3.11.5.6 and C11.8.6.3).

$$f_p = \text{ smallest of } \begin{cases} 0.08\phi_f' = 0.08(35°) = 2.80, & \\ 3, or & = 2.8 \\ \dfrac{S}{b} = \dfrac{8}{2} = 4 & \end{cases}$$

Note: $f_p b$ should not exceed pile spacing, S.

$$P_p = 0.5 p_p D_0 b f_p = 0.5(0.461 D_0)(D_0)(2)(2.8) = 1.291 D_0^2 \text{ kip}$$

Factored earth force due to LS using 8 feet pile spacing:

$$P_{LS} = \Delta_p HS = 0.068(15)(8) = 8.16 \text{ kip}$$

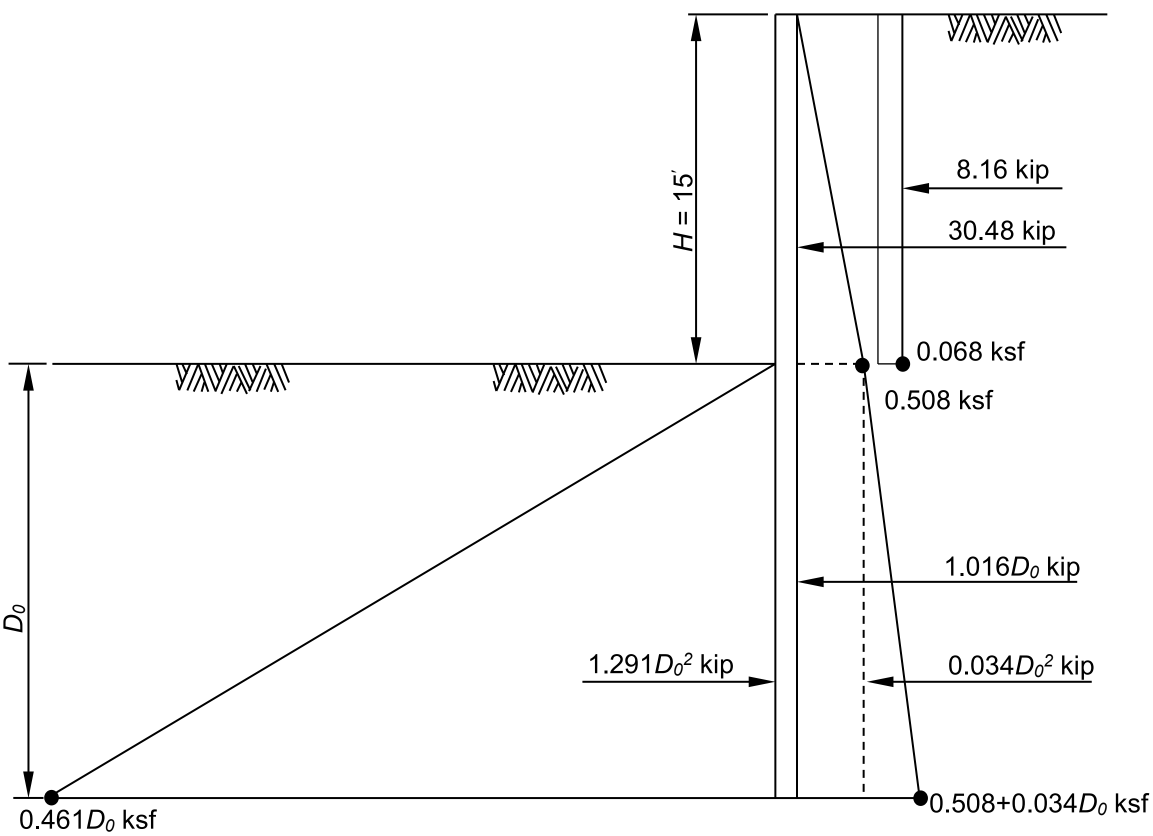


Figure 11.2.3-2 Force Diagram for Service I Limit State

- Calculate factored driving and resisting moments as shown in Table 11.2.3-6, about D_0

Table 11.2.3-6 Service I Limit State Factored Driving & Resisting Moments

Driving Force (kip)	Arm (ft)	Driving Moment, M_{DR} (k-ft)
8.16	$7.5+D_0$	$61.2+8.16D_0$
30.48	$5+D_0$	$152.4+30.48D_0$
$1.016D_0$	$\frac{D_0}{2}$	$0.508D_0^2$
$0.034D_0^2$	$\frac{D_0}{3}$	$0.011D_0^3$
Resisting Force (kip)	Arm (ft)	Resisting Moment, M_{RS} (k-ft)
$1.291D_0^2$	$\frac{D_0}{3}$	$0.430D_0^3$

$$M_{DR}=0.011D_0^3+0.508D_0^2+30.48D_0+152.4+8.16D_0+61.2$$

$$M_{RS}=0.43D_0^3$$

- Calculate pile embedment D_0 required to provide moment equilibrium

$$M_{RS}=M_{DR}$$

$$-0.419D_0^3+0.508D_0^2+38.64D_0+213.6=0$$

$$D_0^3-1.212D_0^2-92.22D_0-509.79=0$$

$$D_0=12.20\text{ ft}$$

Note: In Service I Limit State, D_0 value is needed in order to calculate pile deflection. For walls sensitive to deflections, design embedment D may need to be increased.

- Calculate deflection at the top of the soldier pile using the Moment Area Method

The following steps are needed to calculate top of wall deflection:

Calculate the combined active and passive pressure distribution along the length of the soldier pile.

Integrate the pressure distribution to calculate the shear distribution along the length of the soldier pile.

$$V=\int p\,dl$$

Integrate the shear distribution to calculate the moment distribution along the length of the soldier pile.

$$M = \int V dl$$

Integrate the moment distribution to calculate the slope distribution along the length of the soldier pile.

$$\theta = \int \frac{M}{EI} dl$$

Integrate the slope distribution to calculate the deflection along the length of the soldier pile.

$$y = \int \theta dl$$

where:

$p =$ pressure at any point

$V =$ shear force at any point

$M =$ moment at any point

$\theta =$ slope at any point

$dl =$ incremental length along the pile

$y =$ deflection at any point

Note: For deflection example calculations, see Example 6-3 of T&S, 2011.

Figure 11.2.3-3 represents the deflected shape of a W14x120 soldier pile based on the Moment Area method, using CT-Flex:

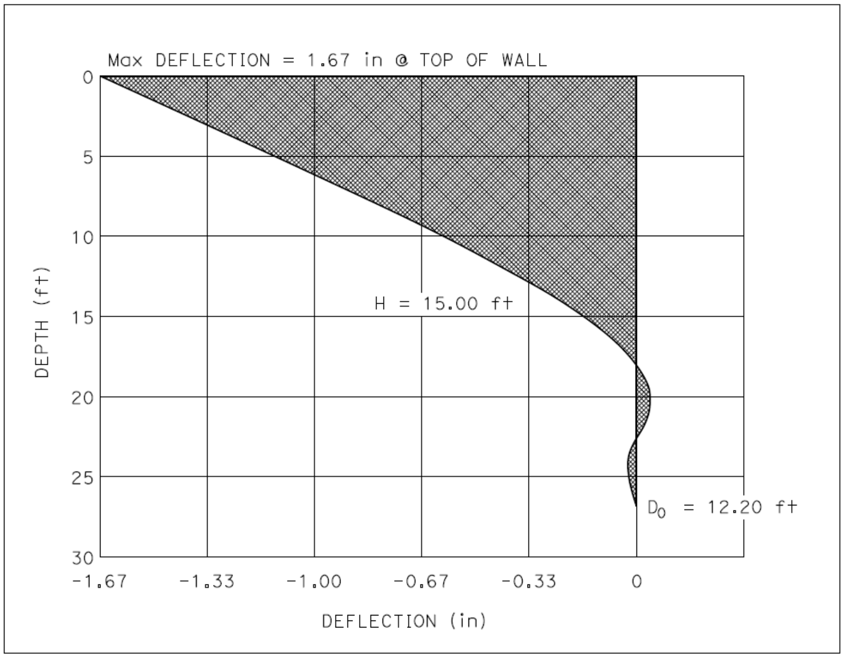


Figure 11.2.3-3 Service I Limit State Deflection

- Check soldier pile deflection

  Soldier pile deflection limits are determined according to the importance of the wall, the location of the wall, and the sensitivity of the facilities behind the wall to wall movements. The deflection limits should be discussed at the Type Selection meeting with the Project Development Team. For walls sensitive to deflections, a more sophisticated analysis such as Finite Element Analysis or Beam-Spring model should be performed, and the pile embedment *D* required to limit these deflections may control.

- Calculate the total factored uniform load, W_{lag}, for a single lagging at the excavation line

  For Service I Limit State, try full sawn 4x12 Douglas Fir-Larch No. 1 lagging with *I* = 64 in.^{4} and *E* = 1,700 ksi.

  Facing may be designed assuming simple support between elements, with soil arching, f_a (Refer to AASHTO Article C11.8.5.2).

$$f_a = \frac{0.083pL^2}{0.125pL^2} = 0.664$$

$$W_{lag} = f_a\left(p_a + \Delta_p\right)\left(\frac{w_b}{12}\right) = 0.664\left(0.508 + 0.068\right)\left(\frac{12}{12}\right) \approx 0.382 \text{ kip/ft}$$

where:

w_b = bearing width = 12 in. for full sawn 4x12 lagging

- Calculate the bearing length required for lagging resistance in compression perpendicular to the grain

Factored reaction, R_u, as shown in Figure 11.2.3-4:

$$R_u = \frac{W_{lag}\left(L_{clear} + \frac{l_b}{12}\right)}{2}$$

Factored resistance in compression perpendicular to the grain:

$$\phi P_n \geq R_u$$

$$P_n = F_{cp} A_b C_b \qquad \text{(AASHTO 8.8.3-1)}$$

$$A_b = l_b w_b$$

By substitution:

$$\phi\left(F_{cp}\, l_b w_b C_b\right) = \frac{W_{lag}\left(L_{clear} + \frac{l_b}{12}\right)}{2}$$

$$l_b = \frac{W_{lag} L_{clear}}{2\phi\left(F_{cp} w_b C_b\right) - \frac{W_{lag}}{12}} = \frac{0.382(7.5)}{2\,(1.0)(0.45)(12)(1.0) - \frac{0.382}{12}} = 0.27 \text{ in.}$$

where:

l_b = required bearing length (in.)

L_{clear} = clear span length between flanges = 7.5 ft (Assumed length that may be adjusted after pile size selection)

ϕ = resistance factor of wood in compression perpendicular to grain = 1.0 (Refer to Table 11.2.3-3)

F_{cp} = adjusted design value of wood in compression perpendicular to grain = 0.45 ksi (Refer to Table 11.2.3-4)

C_b = bearing adjustment factor = 1.0 (Refer to AASHTO Article 8.8.3)

Note: In Service I Limit State, l_b value is only needed in order to determine lagging deflection.

- Calculate the lagging span length, L

$$L = L_{clear} + \frac{l_b}{12}$$

$$L = 7.5 + \frac{0.27}{12} = 7.52 \text{ ft}$$

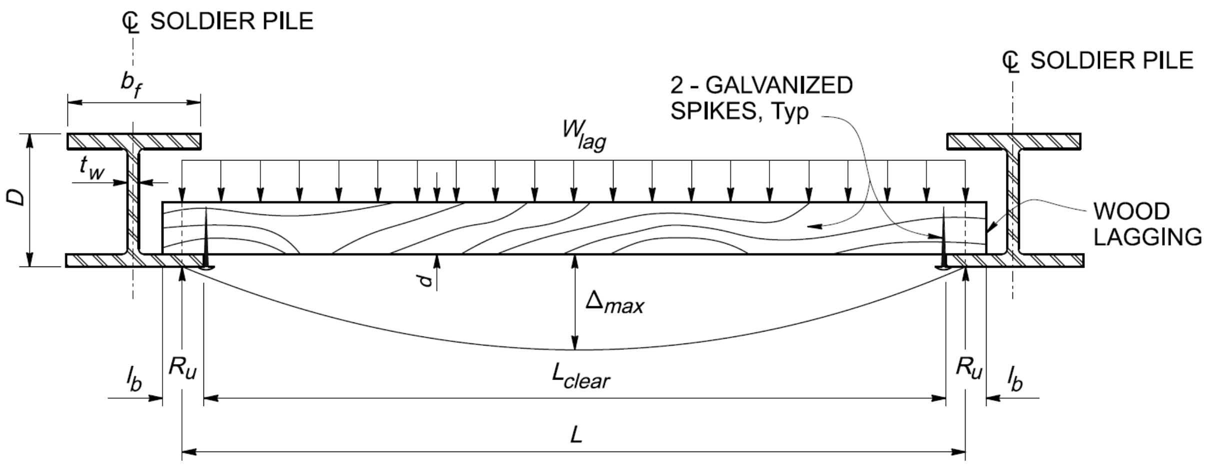


Figure 11.2.3-4 Lagging Parameters

- Calculate maximum lagging deflection

$$\Delta_{max} = \frac{5(W_{lag}/12)(12L)^4}{384EI} = \frac{5(0.382/12)[12(7.52)]^4}{384(1{,}700)(64)} = 0.253 \text{ in.}$$

- Check wood lagging deflection

  Wood lagging deflection limits are determined according to the importance of the wall, location of the wall, and the sensitivity of the facilities behind the wall to wall movements. The deflection limits should be discussed at the Type Selection meeting with the Project Development Team. Deflection limits for wood lagging have not been established in the AASHTO code, but based on past practice, an allowable deflection of L/240 is a good rule of thumb.

11.2.3.2.2 Overall Stability (Refer to AASHTO Article 11.9.3.2)

For Service, I Limit State overall stability, a slope stability analysis is done by Geotechnical Services.

11.2.3.3 Strength I Limit State Checks

11.2.3.3.1 Overall Stability (Refer to AASHTO Article 11.8.4.1)

For Strength I Limit State overall stability, use the following procedure:

a. Determine pile embedment *D* required to ensure stability against passive failure;
b. Determine pile embedment *D* required to ensure stability against slope failure (done by Geotechnical Services);
c. Use the greater of the two values determined in the steps above.

The following calculations are needed as intermediate steps in order to determine pile embedment *D* required to ensure stability against passive failure:

- Calculate active and passive earth pressure coefficients

$k_a = 0.271$ (Refer to Service I Limit State)

$k_p = 3.690$ (Refer to Service I Limit State)

- Calculate factored lateral earth pressure distributions

Factored active *EH* at excavation line:

$$p_a = k_a \gamma_s H \gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_a = 0.271(0.125)(15)(1.5) = 0.762 \text{ ksf}$$

where:

$\gamma_p =$ active EH load factor $= 1.5$ (Refer to Table 11.2.3-1)

Factored active *EH* at depth D_o:

$$p_{aD_0} = k_a \gamma_s \left(H + D_0\right) \gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_{aD_0} = 0.271(0.125)(15 + D_0)(1.5) = 0.762 + 0.051 D_0 \text{ ksf}$$

Factored passive lateral earth pressure at depth D_o:

$$p_p = k_p \gamma_s D_0 \phi \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_p = 3.69(0.125 D_0)(1.0) = 0.461 D_0 \text{ ksf}$$

where:

$\phi =$ passive lateral earth pressure resistance factor $= 1.0$ (Refer to Table 11.2.3-2)

Factored lateral earth pressure due to *LS* above the excavation line only:

$\Delta_p = k_a \gamma_s h_{eq} \gamma$ (Per AASHTO 3.11.6.4-1)

$\Delta_p = 0.271(0.125)(2)(1.75) = 0.119$ ksf

where:

$h_{eq} =$ equivalent height of soil for vehicular load $= 2$ ft (Refer to AASHTO Table 3.11.6.4-2)

$\gamma = LS$ load factor $= 1.75$ (Refer to Table 11.2.3-1)

- Calculate factored resultant earth forces for a single pile (Refer to AASHTO Article 3.11.5.6)

Factored active earth force above excavation line using 8 feet pile spacing:

$P_{a1} = 0.5 p_a HS = 0.5(0.762)(15)(8) = 45.72$ kip

Factored active earth forces below the excavation line using 2 feet diameter hole:

$P_{a2_1} = p_a D_0 b = 0.762 D_0 (2) = 1.524 D_0$ kip

$P_{a2_2} = 0.5(0.051 D_0)(D_0)(2) = 0.051 D_0^2$ kip

Factored passive earth force below the excavation line using 2 feet diameter hole and soil arching capability factor:

$P_p = 1.291 D_0^2$ kip (Refer to Service I Limit State)

Factored earth force due to *LS* using 8 feet pile spacing:

$P_{LS} = \Delta_p HS = 0.119(15)(8) = 14.28$ kip

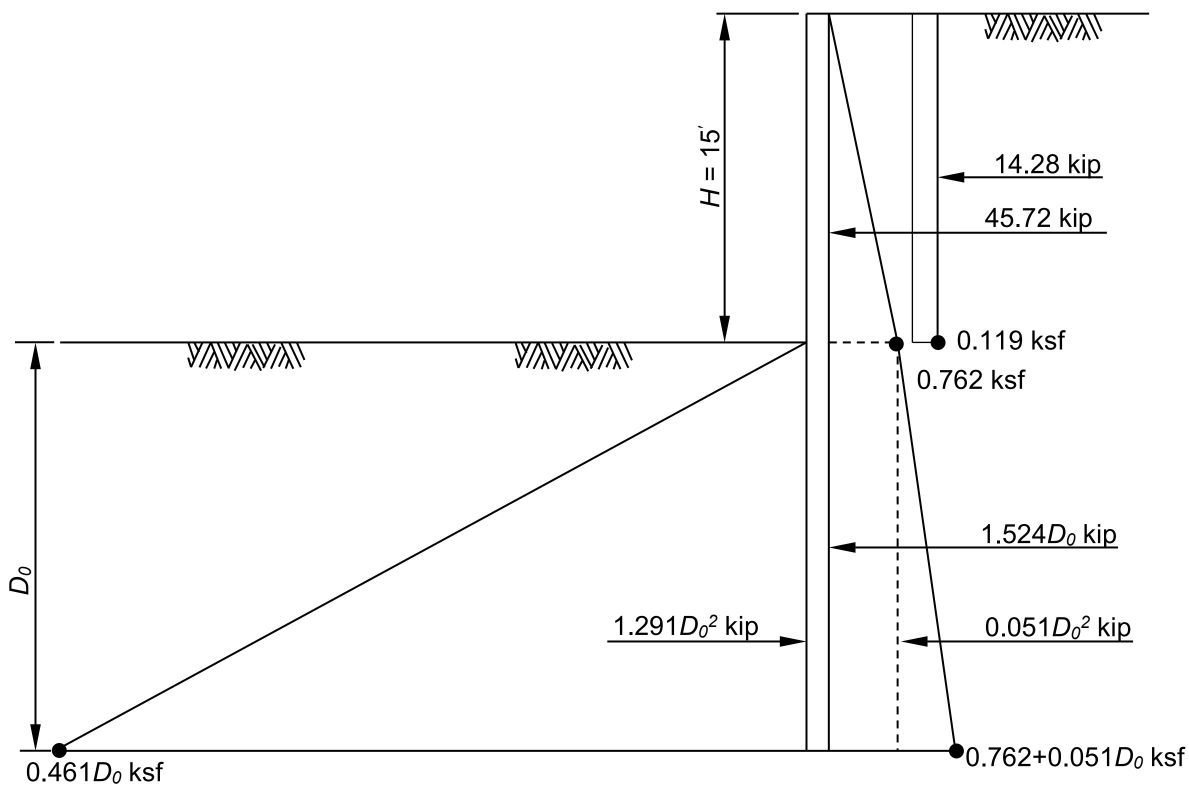


Figure 11.2.3-5 Force Diagram for Strength I Limit State

- Calculate factored driving and resisting moments as shown in Table 11.2.3-7, about D_0

Table 11.2.3-7 Strength I Limit State Factored Driving & Resisting Moments

Driving Force (kip)	Arm (ft)	Driving Moment, M_{DR} (k-ft)
14.28	$7.5+D_0$	$107.1+14.28D_0$
45.72	$5+D_0$	$228.6+45.72D_0$
$1.524D_0$	$\frac{D_0}{2}$	$0.762D_0^2$
$0.051D_0^2$	$\frac{D_0}{3}$	$0.017D_0^3$
Resisting Force (kip)	Arm (ft)	Driving Moment, M_{DR} (k-ft)
$1.291D_0^2$	$\frac{D_0}{3}$	$0.430D_0^3$

$$M_{DR}=0.017D_0^3+0.762D_0^2+45.72D_0+228.6+14.28D_0+107.1$$

$$M_{RS}=0.43D_0^3$$

- Determine pile embedment D_0 required to provide moment equilibrium and design embedment D (Refer to AASHTO Article C11.8.4.1 and T&S Section 6.1)

  $$M_{RS} = M_{DR}$$

  $$-0.413D_0^3 + 0.762D_0^2 + 60D_0 + 335.7 = 0$$

  $$D_0^3 - 1.845D_0^2 - 145.278D_0 - 812.833 = 0$$

  $$D_0 = 15.07 \text{ ft}$$

  $D = 1.2D_0 = 1.2 \times 15.07 = 18.08 \text{ ft} \rightarrow \text{Try } 19 \text{ ft}$ (Compare with pile embedment D provided by Geotechnical Services, and use controlling D)

  Note: Another acceptable method for calculating pile embedment D provides both shear and moment equilibrium at the pile tip.

11.2.3.3.2 Vertical Wall Elements (Refer to AASHTO Article 11.8.5.1)

For Strength I Limit State, try W14x120 Grade 50 steel beam with E = 29,000 ksi.

The following calculations are needed as intermediate steps in order to check the factored flexural and shear resistances of the piles:

- Calculate the maximum factored moment and shear in the piles

  The maximum moment is located at distance Y below the excavation line, where the shear is equal to zero. Therefore, the summation of horizontal forces at a distance Y is set equal to zero.

  $$\sum F_x = 0$$

  $$-1.291Y^2 + 0.051Y^2 + 1.524Y + 45.72 + 14.28 = 0$$

  $$-1.24Y^2 + 1.524Y + 60.00 = 0$$

  $$Y = 7.60 \text{ ft (below the excavation line)}$$

  $$M_{max} = -1.291Y^2\left(\frac{Y}{3}\right) + 0.051Y^2\left(\frac{Y}{3}\right) + 1.524Y\left(\frac{Y}{2}\right) + 45.72(Y+5) + 14.28(Y+7.5)$$

  $$M_{max} = -1.291(7.60)^2\left(\frac{7.60}{3}\right) + 0.051(7.60)^2\left(\frac{7.60}{3}\right) + 1.524(7.60)\left(\frac{7.60}{2}\right)$$
  $$+45.72(7.60+5) + 14.28(7.60+7.5)$$

  $$M_{max} = 654.27 \text{ k-ft}$$

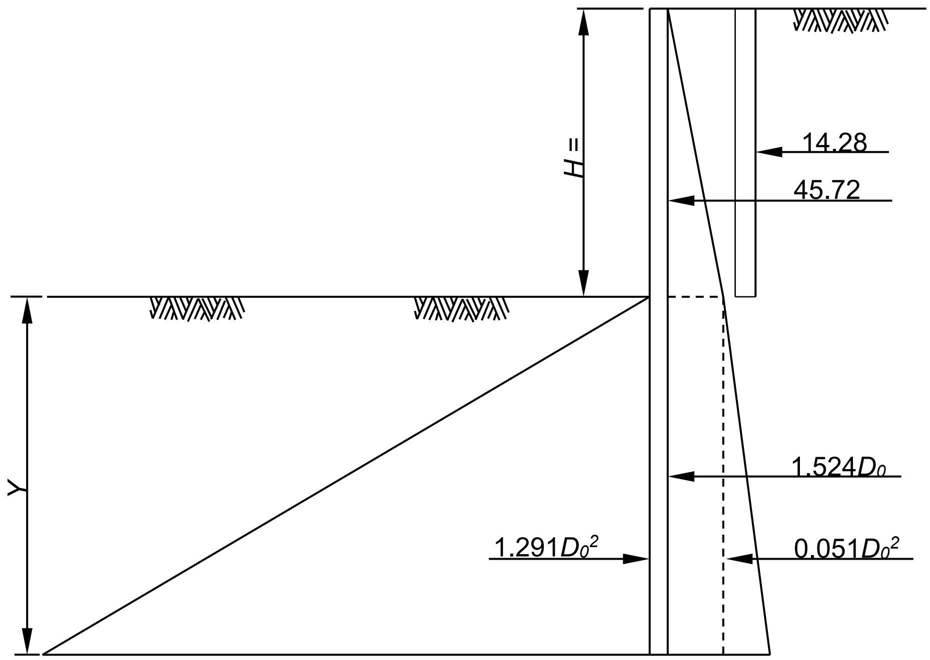


Figure 11.2.3-6 Location of Zero Shear and Maximum Moment for Strength I Limit State

The maximum shear is located at depth D_o:

$$V_{max} = -1.24 D_0^2 + 1.524 D_0 + 60.00$$

$$V_{max} = -1.24(15.07)^2 + 1.524(15.07) + 60.00$$

$$V_{max} = 198.64 \text{ kip}$$

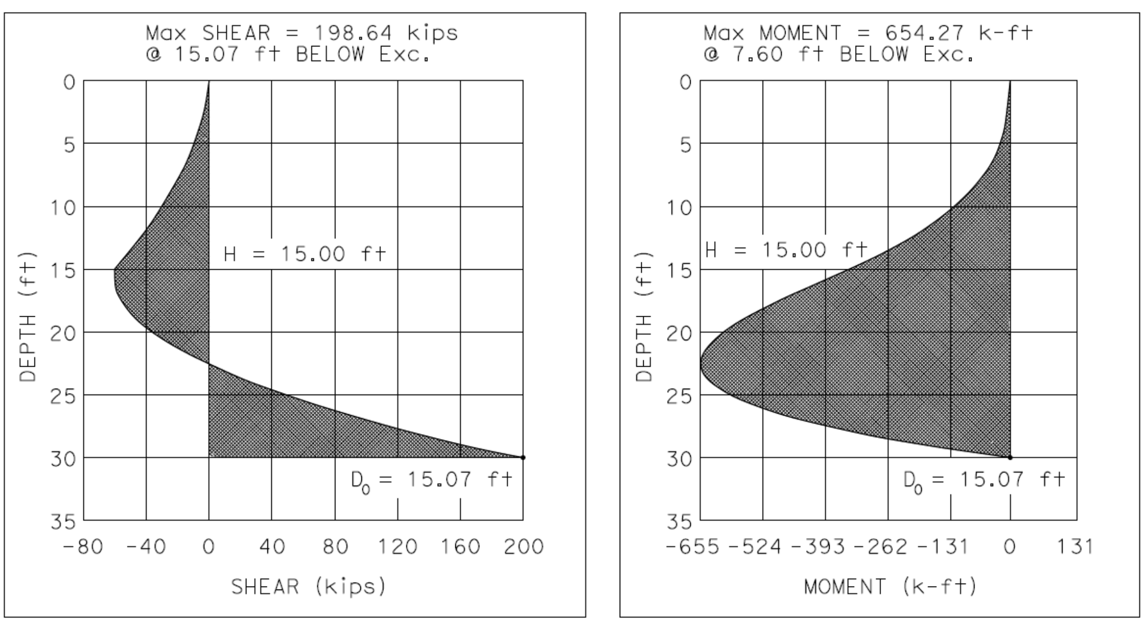


Figure 11.2.3-7 Factored Shear and Moment Diagrams for Strength I Limit State

- Check factored flexural resistance of the piles

  Factored flange stress, f_{bu}:

  $$f_{bu} = \frac{12M_{max}}{S_x} = \frac{12(654.27)}{190} = 41.32 \text{ ksi}$$

  where:

  S_x = section modulus = 190 in.^{3} for W14x120

  Factored flexural resistance:

  Since the location of maximum moment is in the embedded portion of the pile, use the equation for continuously braced flanges.

  $\phi_f R_h F_{yf} \geq f_{bu}$ (AASHTO 6.10.8.1.3-1)

  $0.9(1.0)(50) = 45 \text{ ksi} > 41.32 \text{ ksi}$ OK

  where:

  ϕ_f = steel resistance factor for flexure = 0.9 (Refer to Table 11.2.3-5)

  R_h = hybrid factor = 1.0 (Refer to AASHTO Article 6.10.1.10.1)

  F_{yf} = specified minimum yield strength of the flange = 50 ksi

- Check factored shear resistance of the piles

  Factored shear force, V_u:

  $V_u = V_{max} = 198.64$ kip

  Factored shear resistance of unstiffened webs:

  $\phi_v V_n \geq V_u$ (AASHTO 6.10.9.1-1)

  $V_n = V_{cr} = CV_p$ (AASHTO 6.10.9.2-1)

  $V_p = 0.58 F_{yw} D t_w$ (AASHTO 6.10.9.2-2)

  By substitution:

  $\phi_v \left(C0.58F_{yw}Dt_w\right) = 1.0(1.0)(0.58)(50)(14.48)(0.59) = 247.75 \text{ kip} > 198.64 \text{ kip}$ OK

where:

ϕ_v = steel resistance factor for shear = 1.0 (Refer to Table 11.2.3-5)

C = ratio of the shear buckling resistance to the shear yield strength = 1.0 (Refer to AASHTO Eq. 6.10.9.3.2-4)

F_{yw} = specified minimum yield strength of the web = 50 ksi

D = web depth = 14.48 in. for W14x120

t_w = web thickness = 0.59 in. for W14x120

Note: In this example, a W14x120 soldier pile would work for Strength I Limit State.

11.2.3.3.3 Facing (Refer to AASHTO Article 11.8.5.2)

For Strength I Limit State, try full sawn 4x12 Douglas Fir-Larch No. 1 lagging.

The following calculations are needed as intermediate steps in order to determine the required bearing length and check the factored flexural and shear resistances of the wood lagging:

- Calculate the total factored uniform load, W_{lag}, for a single lagging at the excavation line

$$W_{lag} = f_a\left(p_a + \Delta_p\right)\left(\frac{w_b}{12}\right) = 0.664\left(0.762 + 0.119\right)\left(\frac{12}{12}\right) \approx 0.585 \text{ kip/ft}$$

where:

f_a = soil arching factor = 0.664 (Refer to Service I Limit State)

w_b = bearing width = 12 in. for full sawn 4x12 lagging

- Determine the bearing length required for lagging resistance in compression perpendicular to the grain (Refer to Service I Limit State)

$$l_b = \frac{W_{lag}L_{clear}}{2\phi\left(F_{cp}w_bC_b\right) - \dfrac{W_{lag}}{12}} = \frac{0.585(6.78)}{2\ (0.9)(0.45)(12)(1.0) - \dfrac{0.585}{12}}$$

$l_b = 0.41$ in. $\rightarrow$ Use 3 in. min, per 'Soldier Pile Wall Lagging Details' XS sheet

where:

$$L_{clear} = S - \frac{b_f}{12} = 8 - \frac{14.67}{12} = 6.78 \text{ ft}$$

b_f = flange width of soldier pile = 14.67 in. for W14x120

$\phi = 0.9$ (Refer to Table 11.2.3-3)

$F_{cp} = 0.45$ ksi (Refer to Table 11.2.3-4)

C_b = bearing adjustment factor = 1.0 (Refer to AASHTO Article 8.8.3)

Note: Once the bearing length is determined, check the soldier pile flange width for the minimum constructability requirement, $b_{fmin} \leq b_f$. With no clipping of the lagging corners $b_{fmin} = max(l_b, 3")(2)(2) + t_w = 3(2)(2) + 0.59 = 12.59 \text{ in.} < 14.67 \text{ in. OK}$

- Calculate the lagging span length, L

$$L = L_{clear} + \frac{l_b}{12}$$

$$L = 6.78 + \frac{0.41}{12} = 6.81 \text{ ft}$$

- Check factored flexural resistance of the lagging

Factored moment, M_u:

$$M_u = \frac{W_{lag} L^2}{8} = \frac{0.585(6.81)^2}{8} = 3.39 \text{ k-ft}$$

Factored flexural resistance:

$$\phi M_n \geq M_u$$

$$M_n = \left(F_b S_x C_L\right) / 12 \qquad \text{(Per AASHTO 8.6.2-1)}$$

By substitution:

$$\phi(F_b S_x C_L / 12) = 0.85(1.5)(32)(1.0) / 12 = 3.40 \text{ k-ft} > 3.39 \text{ k-ft OK}$$

where:

ϕ = resistance factor of wood in flexure = 0.85 (Refer to Table 11.2.3-3)

F_b = adjusted design value of wood in flexure = 1.5 ksi (Refer to Table 11.2.3-4)

S_x = 32 in.^{3} for full sawn 4x12 lagging

C_L = stability factor = 1.0 (Refer to AASHTO Article 8.6.2)

- Check factored shear resistance of the lagging

  Factored shear force, V_u, at distance d from the face of support, as defined in AASHTO Article 8.7 (Refer to Figure 11.2.3.4):

$$V_u = W_{lag}\left[\frac{L}{2} - \frac{l_b}{2(12)} - \frac{d}{12}\right] = 0.585\left[\frac{6.81}{2} - \frac{0.41}{2(12)} - \frac{4}{12}\right] = 1.79 \text{ kip}$$

where:

d = lagging depth = 4 in. for full sawn 4x12 lagging

Factored shear resistance:

$$\phi V_n \geq V_u$$

$$V_n = F_v w_b d / 1.5 \qquad \text{(Per AASHTO 8.7-2)}$$

By substitution:

$$\phi(F_v w_b d / 1.5) = 0.75(0.14)(12)(4)/1.5 = 3.36 \text{ kip} > 1.79 \text{ kip OK}$$

where:

ϕ = resistance factor of wood in shear = 0.75 (Refer to Table 11.2.3-3)

F_v = adjusted design value of wood in shear = 0.14 ksi (Refer to Table 11.2.3-4)

Note: In this example, a 4x12 wood lagging would work for Strength I Limit State; however, a 6x12 wood lagging is more commonly used.

11.2.4 ANCHORED SOLDIER PILE WALL DESIGN EXAMPLE

11.2.4.1 Anchored Soldier Pile Wall Data

A multiple ground anchor soldier pile wall with piles 8-foot on center, encased in 2-foot diameter drilled holes, filled with concrete backfill below timber lagging (wood lagging), and filled with lean concrete in the area of wood lagging. The ground anchors are on a 15° inclination angle, with 8-foot horizontal and 10-foot vertical spacing. Soil properties are shown in Figure 11.2.4-1.

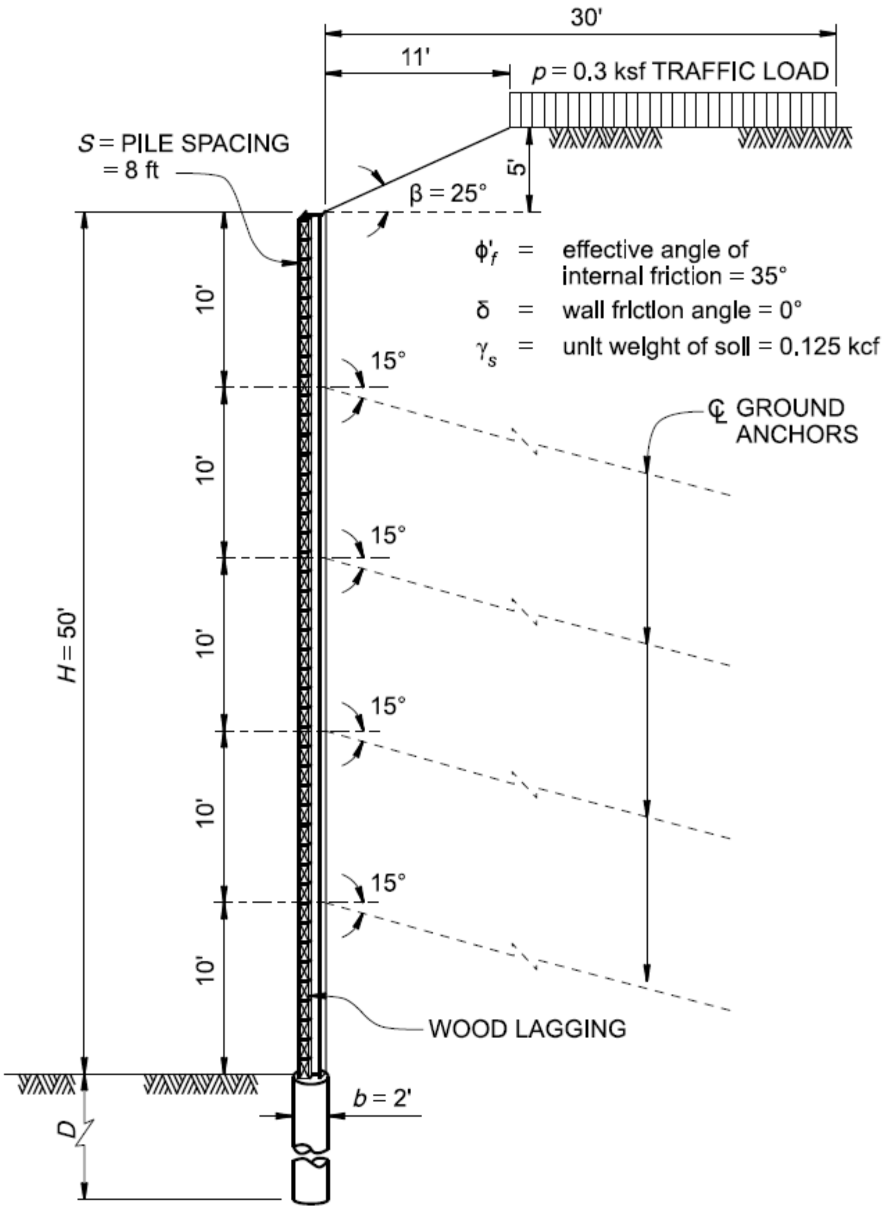


Figure 11.2.4-1 Multiple Ground Anchor Soldier Pile Wall

Perform the design calculations for the anchored soldier pile wall using the Modified Hinge method in accordance with the AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019), and the Caltrans Trenching and Shoring Manual (T&S) (Caltrans, 2011). Also calculate the forces in the ground anchors and the embedment depth using the Hinge method. Perform design calculations for the following limit states:

- Service I Limit State
- Strength I Limit State

For additional considerations, see BDP Section 11.2.3 "Cantilevered Soldier Pile Wall Design Example".

Table 11.2.4-1 summarizes the load factors for the target limit states:

Table 11.2.4-1 Horizontal Earth Pressure (*EH*) and Live Load Surcharge (*LS*) Load Factors

Limit State	*AEP* and Active *EH**	*LS*	Reference
Service I	1.0	1.0	AASHTO Table 3.4.1-2 and CA Table 3.4.1-1
Strength I	1.35	1.75	

*Use the same load factor for both apparent earth pressure (*AEP*) above the excavation line and active *EH* below the excavation line.

Passive lateral earth pressure is considered a resistance and not a load, with the following resistance factors, ϕ:

Table 11.2.4-2 Passive Lateral Earth Pressure Resistance Factors

Limit State	Passive Lateral Earth Pressure	Reference
Service I	1.0	AASHTO Article 11.5.7
Strength I	1.0	CA Table 11.5.7-1

The following resistance factors, ϕ, are used for wood lagging analysis:

Table 11.2.4-3 Wood Lagging Resistance Factors

Limit State	Flexure	Shear	Compression Perpendicular to Grain	Reference
Service I	N/A	N/A	1.0	AASHTO Article 8.5.2.2
Strength I	0.85	0.75	0.9	

The adjusted design values for wood members used in this example problem are shown below:

Table 11.2.4-4 Wood Lagging Adjusted Design Values

Limit State	Flexure	Shear	Compression Perpendicular to Grain	Reference
Service I	N/A	N/A	0.45 ksi	AASHTO Article 8.4.4.1
Strength I	1.5 ksi	0.14 ksi	0.45 ksi	

Note: In this example, the adjusted design values are based on the maximum allowable stresses listed in the Caltrans Standard Specifications 2015, Section 48-2.02B(3)(b). When further guidance is received from the Earth Retaining Systems Committee on the format conversion factor, C_{kf}, for wood lagging, the designer may calculate the adjusted design values per AASHTO.

The following resistance factors are used for steel pile analysis:

Table 11.2.4-5 Steel Pile Resistance Factors

Limit State	Flexure	Shear	Reference
Strength I	0.9	1.0	CA Table 11.5.7-1 and AASHTO Article 6.5.4.2

In the case of multiple ground anchor walls, the recommended AEP distribution in cohesionless soils is shown in Figure 11.2.4-2.

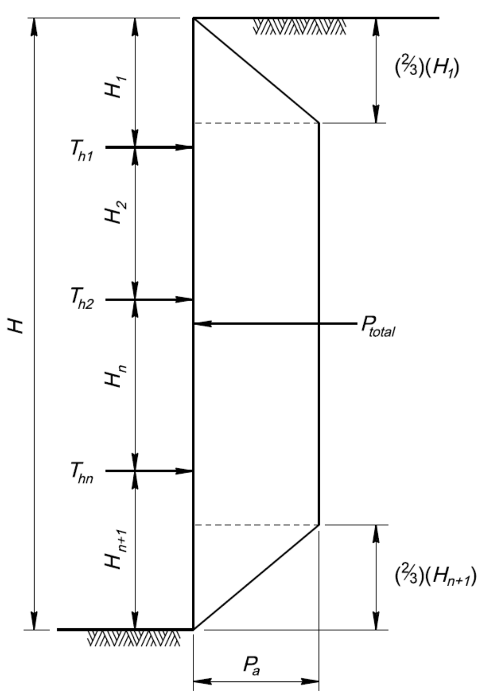


Figure 11.2.4-2 AEP Distribution for Walls with Multiple Levels of Ground Anchors (Refer to AASHTO Figure 3.11.5.7.1-1)

When P_{total} is provided by Geotechnical Services (usually for slide situations):

$$p_a = \frac{P_{total}}{H - \frac{H_1}{3} - \frac{H_{n+1}}{3}}$$

For a single soil layer and no external loading, this equation may be simplified to:

$$p_a = \frac{0.65 k_a \gamma_s H^2}{H - \frac{H_1}{3} - \frac{H_{n+1}}{3}}$$

$$p_a = \frac{k_a \gamma_s H^2}{1.5H - 0.5H_1 - 0.5H_{n+1}}$$ (Per AASHTO 3.11.5.7.1-2)

where:

$p_a =$ maximum ordinate of *AEP* diagram (ksf)

$P_{total} =$ total lateral earth load provided by Geotechnical Services (kip/ft) or $0.65 k_a \gamma_s H^2$ from the rectangular *AEP* diagram for sands developed by Terzaghi and Peck (Refer to Geotechnical Circular No. 4)

H = total excavation depth (ft)

H_1 = distance from the ground surface at the top of the wall to the uppermost level of anchors (ft)

H_{n+1} = distance from the excavation line to the lowermost level of anchors (ft)

n = number of anchors

T_{hn} = horizontal component of the anchor force at level n (kip/ft)

k_a = active earth pressure coefficient

γ_s = unit weight of soil (kcf)

11.2.4.2 Service I Limit State Checks

11.2.4.2.1 Movement (Refer to AASHTO Article 11.9.3.1)

The following calculations are needed as intermediate steps in order to check pile and lagging deflections:

- Calculate active and passive earth pressure coefficients

  Since this example has an irregular backfill condition, use Trial Wedge method to calculate the active earth pressure coefficient, k_a. To simplify the Trial Wedge iterative process, use CT-Flex to calculate k_a (Refer to AASHTO Article A11.3.2 and T&S Section 4.5.1).

$$k_a = 0.310$$

Note: For Trial Wedge example calculations, see Example 4-2 of T&S, 2011.

Since the slope angle in front of the wall and the wall friction angle, δ, are zero in this example, use Rankine's Earth Pressure Theory to calculate the passive earth pressure coefficient, k_p.

$$k_p = \tan^2\left(45 + \frac{\phi_f^{'}}{2}\right) = \tan^2\left(45 + \frac{35}{2}\right) = 3.690 \quad \text{(T\&S 4-10)}$$

Note: Rankine's equation for the passive earth pressure should not be used if the slope angle in front of the wall is greater than zero (Refer to T&S 4.3). Also, Rankine's equation underestimates the passive earth pressure if $\delta > 0$. For passive earth pressure, it is recommended to use Coulomb's Theory (T&S Eq. 4-22) when $0 < \delta \leq 1/3\phi_f^{'}$, and the Log-Spiral Method (AASHTO Figure 3.11.5.4-1) when $\delta > 1/3\phi_f^{'}$. AASHTO Article C3.11.5.3 gives guidance on the determination of δ. In this example, $\delta = 0$ for simplicity.

- Calculate factored lateral earth pressure distributions

Maximum factored AEP above the excavation line:

$$p_a = \frac{k_a \gamma_s H^2 \gamma_p}{1.5H - 0.5H_1 - 0.5H_{n+1}} = \frac{0.310(0.125)(50)^2(1.0)}{1.5(50) - 0.5(10) - 0.5(10)} = 1.490 \text{ ksf}$$

where:

$$\gamma_p = AEP \text{ load factor} = 1.0 \text{ (Refer to Table 11.2.4-1)}$$

Factored active *EH* at excavation line:

$$p_{a1} = k_a \gamma_s H \gamma_p \quad \text{(AASHTO 3.11.5.1-1)}$$

$$p_{a1} = 0.310(0.125)(50)(1.0) = 1.938 \text{ ksf}$$

where:

$$\gamma_p = \text{active } EH \text{ load factor} = 1.0 \text{ (Refer to Table 11.2.4-1)}$$

Factored active *EH* at depth *D*:

$$p_{aD} = k_a \gamma_s \left(H + D\right)\gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_{aD} = 0.310(0.125)(50 + D)(1.0) = 1.938 + 0.039D \text{ ksf}$$

Factored passive lateral earth pressure at depth *D*:

$$p_p = k_p \gamma_s D\phi \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_p = 3.69(0.125D)(1.0) = 0.461D \text{ ksf}$$

where:

$\phi =$ passive lateral earth pressure resistance factor $= 1.0$ (Refer to Table 11.2.4-2)

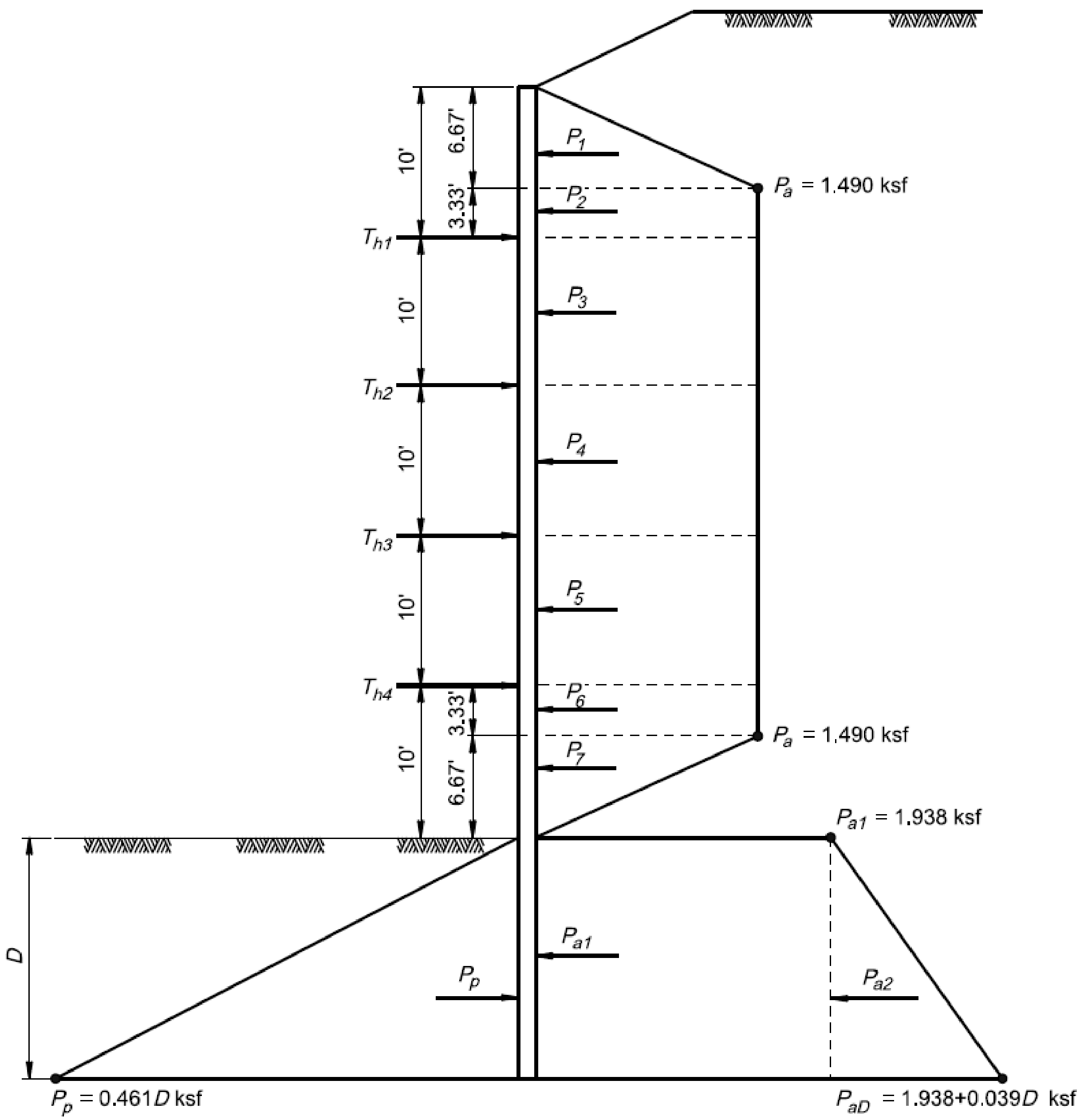


Figure 11.2.4-3 Lateral Earth Pressure Diagram for Service I Limit State

- Calculate factored resultant earth forces for a single pile

  Factored apparent earth forces above excavation line using 8 feet pile spacing:

$$P_1 = P_7 = 0.5 p_a \left(\frac{2}{3}\right) H_1 S = 0.5(1.49)\left(\frac{2}{3}\right)(10)(8) = 39.73 \text{ kip}$$

$$P_2 = P_6 = p_a\left(\frac{1}{3}\right)H_1 S = 1.49\left(\frac{1}{3}\right)(10)(8) = 39.73 \text{ kip}$$

$$P_3 = P_4 = P_5 = p_a H_2 S = 1.49(10)(8) = 119.20 \text{ kip}$$

Factored active earth forces below excavation line using 2 feet diameter hole:

$$P_{a1} = p_{a1} Db = 1.938D(2) = 3.876D \text{ kip}$$

$$P_{a2} = 0.5(0.039D)(D)(2) = 0.039D^2 \text{ kip}$$

Factored passive earth force below the excavation line using 2 feet diameter hole and soil arching capability factor:

For soldier piles, a soil arching capability factor, f_p, needs to be calculated and applied to the passive forces only (Refer to AASHTO Articles C3.11.5.6 and C11.8.6.3).

$$f_p = \text{smallest of} \begin{cases} 0.08\phi'_f = 0.08(35^\circ) = 2.80, \\ 3, or \\ \dfrac{S}{b} = \dfrac{8}{2} = 4 \end{cases} = 2.8$$

Note: $f_p b$ should not exceed pile spacing, S.

$$P_p = 0.5 p_p D b f_p = 0.5(0.461D)(D)(2)(2.8) = 1.291D^2 \text{ kip}$$

- Calculate factored lateral pressure distribution due to *LS*, using the Boussinesq method

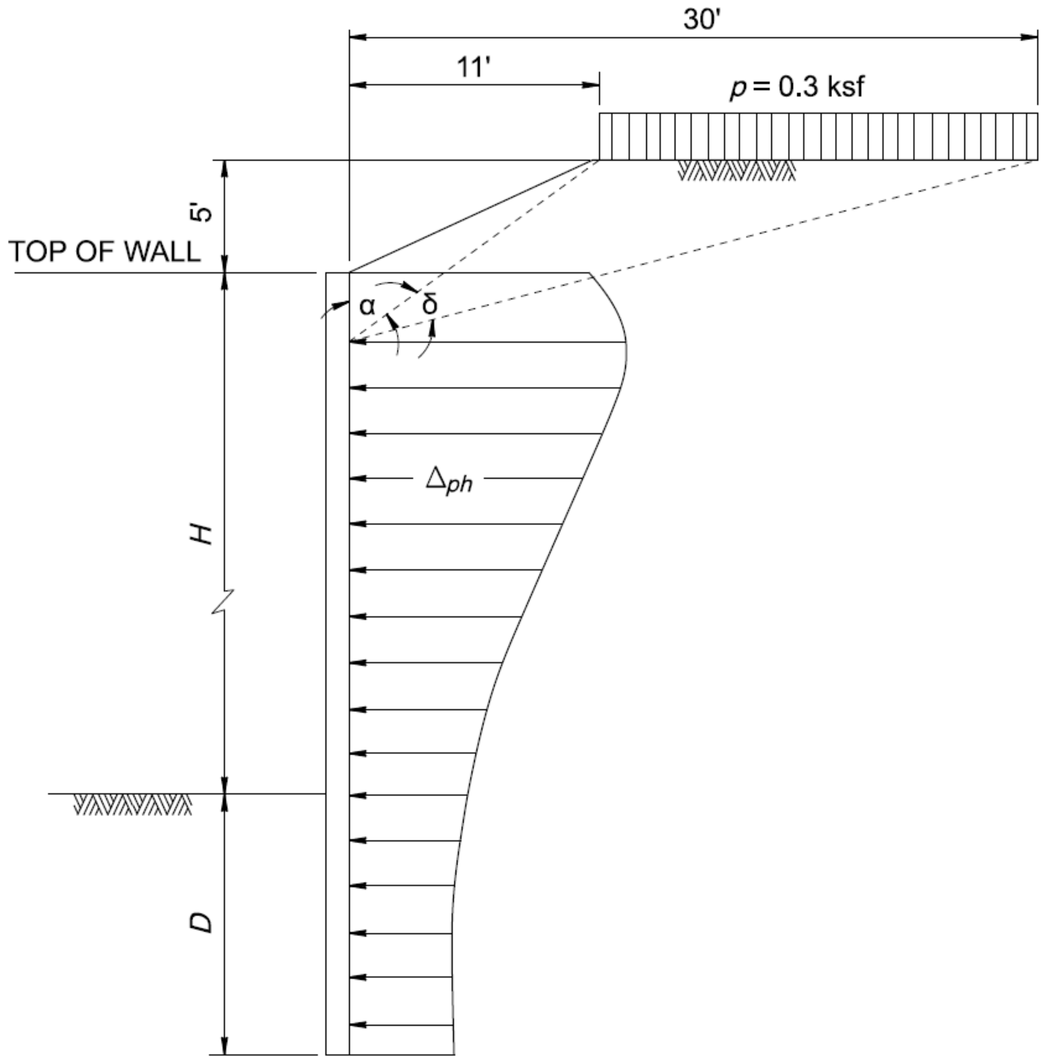


Figure 11.2.4-4 Horizontal Pressure Caused by Strip Load Based on Boussinesq Method (Refer to AASHTO Figure 3.11.6.2-1)

Factored lateral pressure due to *LS*, Δ_{ph}, at top of the wall:

$$\alpha = \tan^{-1}\left(\frac{11}{5}\right) = 1.144 \text{ rad}$$

$$\delta = \tan^{-1}\left(\frac{30}{5}\right) - \tan^{-1}\left(\frac{11}{5}\right) = 0.261 \text{ rad}$$

$$\Delta_{ph} = \frac{2p}{\pi}[\delta - \sin\delta\ \cos(\delta + 2\alpha)]\gamma \qquad \text{(Per AASHTO 3.11.6.2-1)}$$

$$\Delta_{ph} = \frac{2(0.3)}{\pi}\left[0.261 - \sin(0.261)\cos\left(0.261 + 2(1.144)\right)\right]1.0 = 0.091 \text{ ksf}$$

where:

$p =$ uniform strip load parallel to wall $= 0.3$ ksf

$\gamma = LS$ load factor $= 1.0$ (Refer to Table 11.2.4-1)

Table 11.2.4-6 Boussinesq Pressure Values at Various Depths

Depth (ft)	Lateral Pressure (ksf)	Location
0	0.091	Top of wall
5	0.117	5 ft from top of wall
10	0.105	10 ft from top of wall
50	0.014	Excavation line
55	0.011	5 ft below excavation line

Note: Lateral pressure was assumed to remain constant below 55 ft depth since the change was negligible.

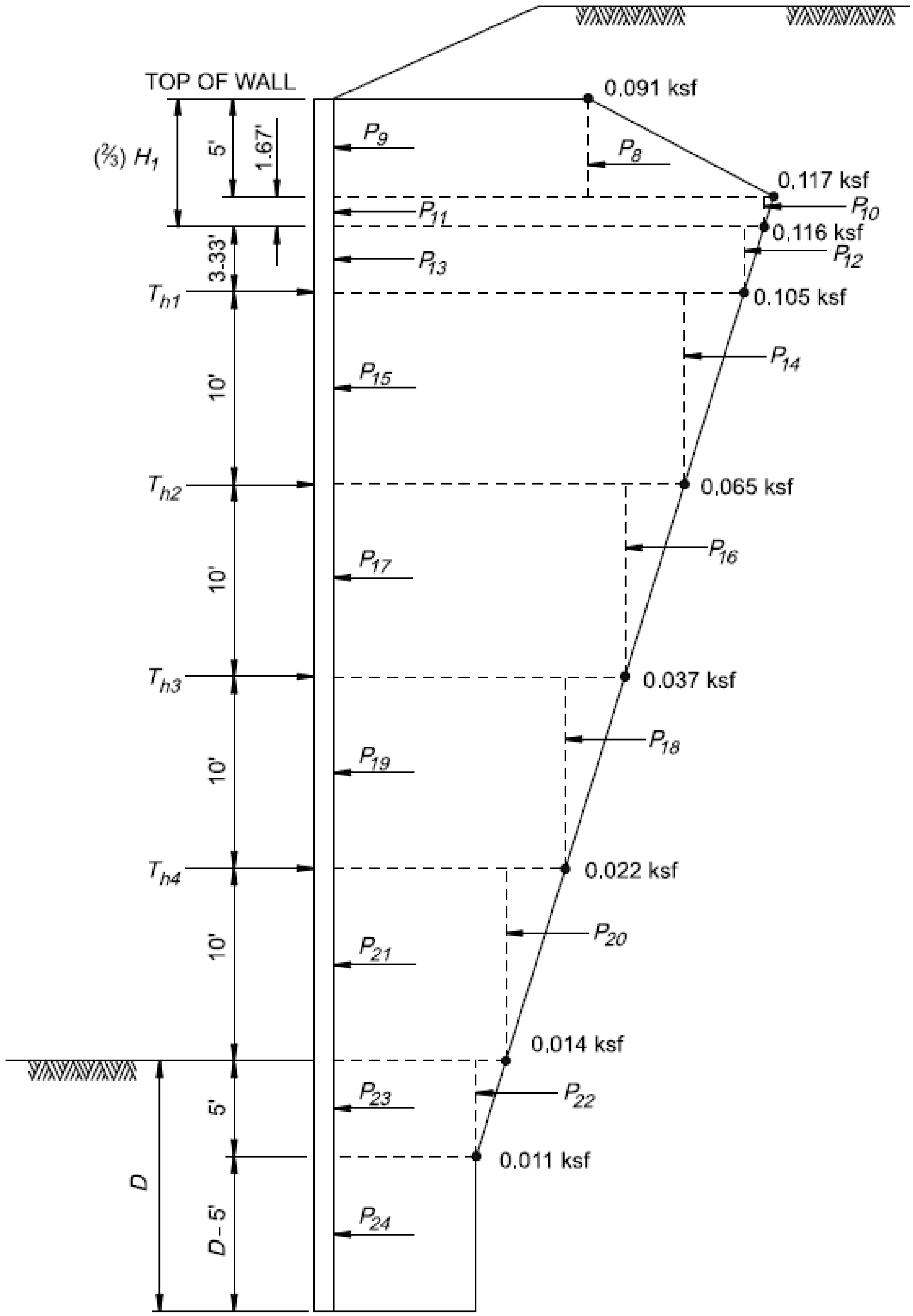


Figure 11.2.4-5 Lateral Pressure Diagram due to *LS* Based on Boussinesq Method for Service I Limit State

- Calculate factored lateral forces due to *LS* for a single pile

Factored lateral forces due to *LS* above excavation line using 8 feet pile spacing:

$$P_8 = 0.5(5)(0.117 - 0.091)(8) = 0.52 \text{ kip}$$

$$P_9 = 0.091(5)(8) = 3.64 \text{ kip}$$

$$P_{10} = 0.5(1.67)(0.117 - 0.116)(8) = 0.01 \text{ kip}$$

$$P_{11} = 0.116(1.67)(8) = 1.55 \text{ kip}$$

$$P_{12} = 0.5(3.33)(0.116 - 0.105)(8) = 0.15 \text{ kip}$$

$$P_{13} = 0.105(3.33)(8) = 2.80 \text{ kip}$$

$$P_{14} = 0.5(10)(0.105 - 0.065)(8) = 1.60 \text{ kip}$$

$$P_{15} = 0.065(10)(8) = 5.20 \text{ kip}$$

$$P_{16} = 0.5(10)(0.065 - 0.037)(8) = 1.12 \text{ kip}$$

$$P_{17} = 0.037(10)(8) = 2.96 \text{ kip}$$

$$P_{18} = 0.5(10)(0.037 - 0.022)(8) = 0.60 \text{ kip}$$

$$P_{19} = 0.022(10)(8) = 1.76 \text{ kip}$$

$$P_{20} = 0.5(10)(0.022 - 0.014)(8) = 0.32 \text{ kip}$$

$$P_{21} = 0.014(10)(8) = 1.12 \text{ kip}$$

Factored lateral forces due to *LS* below the excavation line using 2 feet diameter hole:

$$P_{22} = 0.5(5)(0.014 - 0.011)(2) = 0.02 \text{ kip}$$

$$P_{23} = 0.011(5)(2) = 0.11 \text{ kip}$$

$$P_{24} = 0.011(D - 5)(2) = 0.022D - 0.11 \text{ kip}$$

- Calculate D by taking moments about T_{hnL} (T_{h4L} in this example) for lower section, per the Modified Hinge method

MODIFIED HINGE METHOD (Assume hinges at T_{h2} and T_{hn})

$D = $ Calculated from $\sum M_D = 0$ for DEL

$T_{h1} = $ Calculated from $\sum M_C = 0$

$T_{h2u} = $ Total earth pressure (ABCGF) $- T_{h1}$

$T_{h2L} = $ Calculated from $\sum M_D = 0$

$T_{h2} = T_{h2u} + T_{h2L}$

$T_{hnu} = $ Total earth pressure (CDIH) $- T_{h2L}$

$T_{hnL} = $ Calculated from $\sum F = 0$ for DEL

$= $ Total earth pressure($DEKJ + ELON - EML$)

$T_{hn} = T_{hnu} + T_{hnL}$

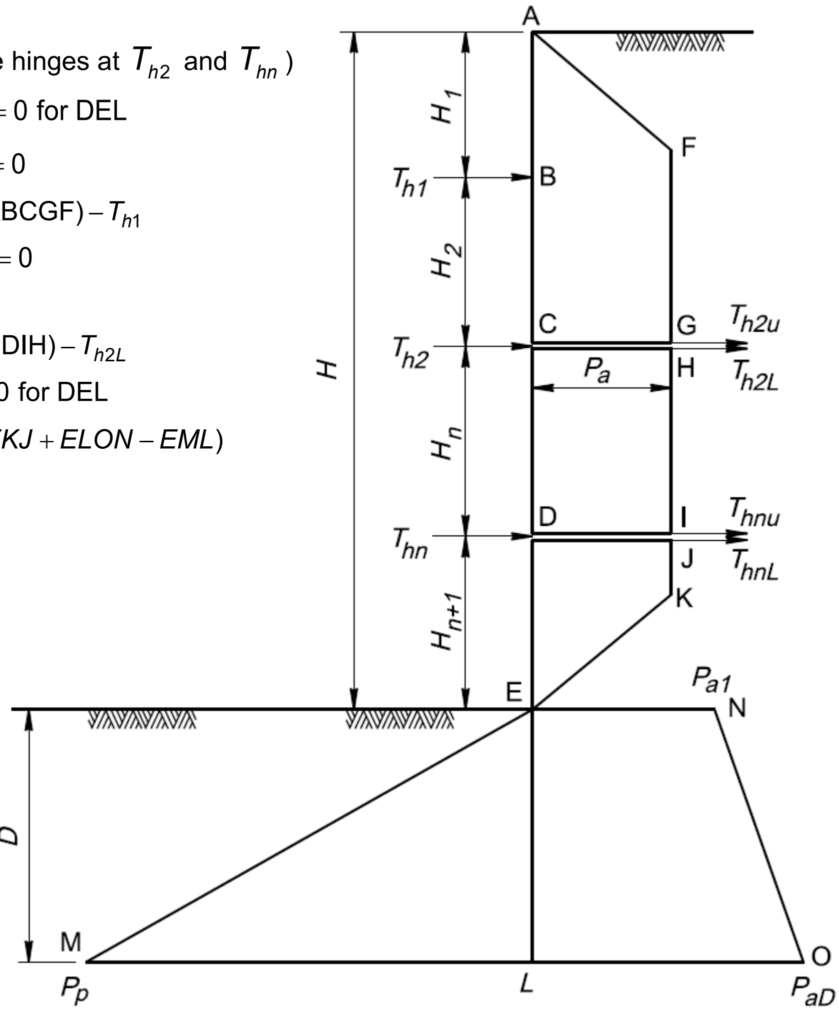


Figure 11.2.4-6 Modified Hinge Method Calculation of Ground Anchor Loads and Pile Embedment for Multilevel Wall (Refer to CA Fig. C11.9.5.1-3)

Note: In this example, the Modified Hinge method has been selected for simplicity to calculate D at the Service I Limit State.

Factored driving and resisting moments, about T_{h4L}, using Modified Hinge method:

Table 11.2.4-7 Service I Limit State Factored Driving & Resisting Moments

Driving Force (kip)	Arm (ft)	Driving Moment, M_{DR} (k-ft)
$P_6 = 39.73$	$\frac{3.33}{2}$	66.15
$P_7 = 39.73$	$\frac{6.67}{3} + 3.33$	220.63
$P_{a1} = 3.876D$	$\frac{D}{2} + 10$	$1.938D^2 + 38.76D$
$P_{a2} = 0.039D^2$	$\frac{2D}{3} + 10$	$0.026D^3 + 0.39D^2$
$P_{20} = 0.32$	$\frac{10}{3}$	1.07
$P_{21} = 1.12$	$\frac{10}{2}$	5.60
$P_{22} = 0.02$	$10 + \frac{5}{3}$	0.23
$P_{23} = 0.11$	$10 + \frac{5}{2}$	1.38
$P_{24} = 0.022D - 0.11$	$10 + \frac{5}{2} + \frac{D}{2}$	$0.011D^2 + 0.22D - 1.38$
Resisting Force (kip)	Arm (ft)	Resisting Moment, M_{RS} (k-ft)
$P_p = 1.291D^2$	$\frac{2D}{3} + 10$	$0.861D^3 + 12.91D^2$

$$M_{DR} = 0.026D^3 + 2.339D^2 + 38.98D + 293.68$$

$$M_{RS} = 0.861D^3 + 12.91D^2$$

Pile embedment D required to provide moment equilibrium:

$$M_{RS} = M_{DR}$$

$$-0.835D^3 - 10.571D^2 + 38.98D + 293.68 = 0$$

$$D^3 + 12.66D^2 - 46.68D - 351.71 = 0$$

$$D = 5.81 \text{ ft}$$

Note: In Service I Limit State, *D* value is needed in order to calculate pile deflection. For walls sensitive to deflections, design embedment *D* may need to be increased.

- Calculate the maximum deflection of the soldier pile using the Moment Area Method

  The process used to calculate soldier pile deflection can be found in BDP Section 11.2.3 and Example 8-1 of T&S (Caltrans, 2011).

  Figure 11.2.4-7 represents the deflected shape of a HP14x117 soldier pile based on the Moment Area method, using CT-Flex:

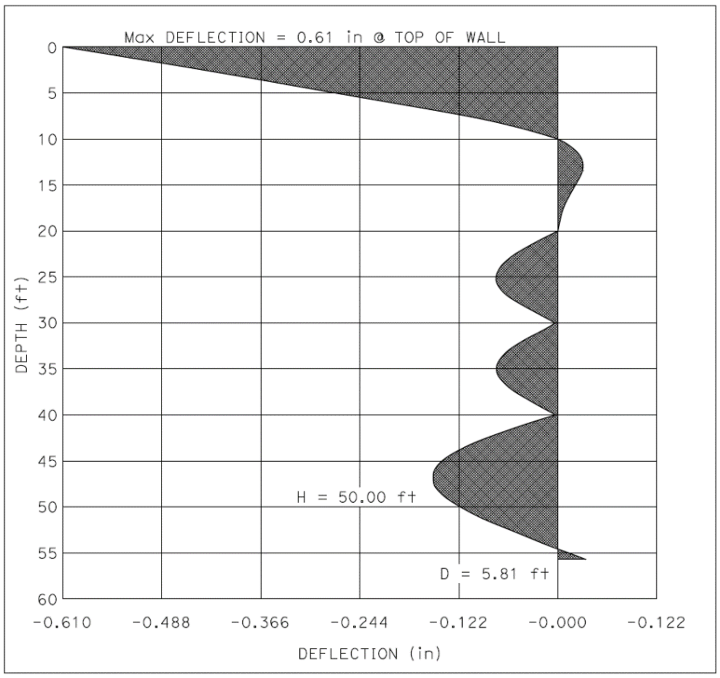


Figure 11.2.4-7 Service I Limit State Deflection

- Check soldier pile deflection

  Soldier pile deflection limits are determined according to the importance of the wall, the location of the wall, and the sensitivity of the facilities behind the wall to wall movements. The deflection limits should be discussed at the Type Selection meeting with the Project Development Team. For walls sensitive to deflections, a more sophisticated analysis such as Finite Element Analysis or Beam-Spring model should be performed, and the pile embedment *D* required to limit these deflections may control.

- Calculate and check wood lagging deflection

  The process to check wood lagging deflection can be found in BDP Section 11.2.3 and will not be shown in this example.

11.2.4.2.2 Overall Stability (Refer to AASHTO Article 11.8.3.2)

For Service I Limit State overall stability, a slope stability analysis is done by Geotechnical Services.

11.2.4.3 Strength I Limit State Checks

11.2.4.3.1 Bearing Resistance (Refer to AASHTO Article 11.9.4.1)

Pile embedment *D* required to ensure bearing resistance against all vertical components of loads is determined by Geotechnical Services.

11.2.4.3.1 Anchor Pullout Capacity (Refer to AASHTO Article 11.9.4.2)

Ground anchor bonded length in soil or rock is determined by the Contractor.

11.2.4.3.2 Passive Resistance (Refer to AASHTO Article 11.9.4.3)

The following calculations are needed as intermediate steps in order to determine pile embedment *D* required to ensure stability against passive failure:

- Calculate active and passive earth pressure coefficients

  $k_a = 0.310$ (Refer to Service I Limit State)

  $k_p = 3.690$ (Refer to Service I Limit State)

- Calculate factored lateral earth pressure distributions

  Maximum factored AEP above the excavation line:

$$p_a = \frac{k_a \gamma_s H^2 \gamma_p}{1.5H - 0.5H_1 - 0.5H_{n+1}} = \frac{0.310(0.125)(50)^2(1.35)}{1.5(50) - 0.5(10) - 0.5(10)} = 2.012 \text{ ksf}$$

  where:

  $\gamma_p = \text{AEP load factor} = 1.35$ (Refer to Table 11.2.4-1)

  Factored active *EH* at excavation line:

$$p_{a1} = k_a \gamma_s H \gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_{a1} = 0.310(0.125)(50)(1.35) = 2.616 \text{ ksf}$$

where:

γ_p = active EH load factor = 1.35 (Refer to Table 11.2.4-1)

Factored active EH at depth D:

$$p_{aD} = k_a \gamma_s \left(H + D\right) \gamma_p \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_{aD} = 0.310(0.125)(50 + D)(1.35) = 2.616 + 0.052D \text{ ksf}$$

Factored passive lateral earth pressure at depth D:

$$p_p = k_p \gamma_s D \phi \qquad \text{(Per AASHTO 3.11.5.1-1)}$$

$$p_p = 3.69(0.125D)(1.0) = 0.461D \text{ ksf}$$

where:

ϕ = passive lateral earth pressure resistance factor = 1.0 (Refer to Table 11.2.4-2)

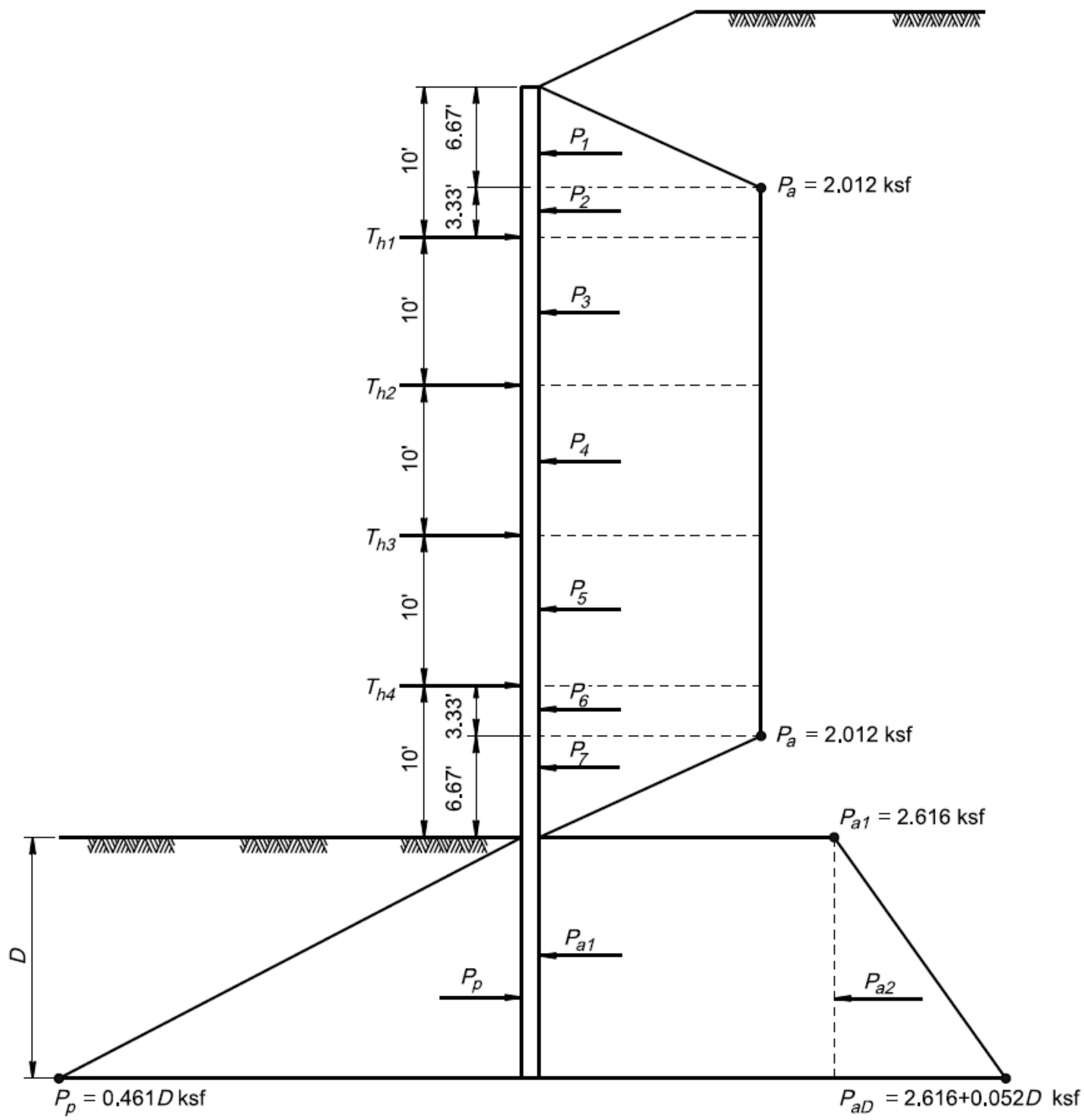


Figure 11.2.4-8 Lateral Earth Pressure Diagram for Strength I Limit State

- Calculate factored resultant earth forces for a single pile

  Factored apparent earth forces above excavation line using 8 feet pile spacing:

$$P_1 = P_7 = 0.5p_a\left(\frac{2}{3}\right)H_1S = 0.5(2.012)\left(\frac{2}{3}\right)(10)(8) = 53.65 \text{ kip}$$

$$P_2 = P_6 = p_a\left(\frac{1}{3}\right)H_1S = 2.012\left(\frac{1}{3}\right)(10)(8) = 53.65 \text{ kip}$$

$$P_3 = P_4 = P_5 = p_aH_2S = 2.012(10)(8) = 160.96 \text{ kip}$$

Factored active earth forces below excavation line using 2 feet diameter hole:

$$P_{a1} = p_{a1}Db = 2.616D(2) = 5.232D \text{ kip}$$

$$P_{a2} = 0.5(0.052D)(D)(2) = 0.052D^2 \text{ kip}$$

Factored passive earth force below the excavation line using 2 feet diameter hole and soil arching capability factor:

$$P_p = 1.291D^2 \text{ kip} \text{ (Refer to Service I Limit State)}$$

- Calculate factored lateral pressure distribution due to *LS* using the Boussinesq method

Factored lateral pressure due to *LS*, Δ_{ph}, at top of the wall (Refer to Figure 11.2.4-4):

$$\alpha = \tan^{-1}\left(\frac{11}{5}\right) = 1.144 \text{ rad}$$

$$\delta = \tan^{-1}\left(\frac{30}{5}\right) - \tan^{-1}\left(\frac{11}{5}\right) = 0.261 \text{ rad}$$

$$\Delta_{ph} = \frac{2p}{\pi}\left[\delta - \sin\delta \cos(\delta + 2\alpha)\right]\gamma \qquad \text{(Per AASHTO 3.11.6.2-1)}$$

$$\Delta_{ph} = \frac{2 \times 0.3}{\pi} \times \left[0.261 - \sin(0.261) \times \cos(0.261 + 2 \times 1.144)\right] \times 1.75 = 0.159 \text{ ksf}$$

where:

p = uniform strip load parallel to wall = 0.3 ksf

$\gamma = LS$ load factor = 1.75 (Refer to Table 11.2.4-1)

Table 11.2.4-8 Boussinesq Pressure Values at Various Depths

Depth (ft)	Lateral Pressure (ksf)	Location
0	0.159	Top of wall
5	0.205	5 ft from top of wall
10	0.184	10 ft from top of wall
50	0.025	Excavation line
55	0.020	5 ft below excavation line

Note: Lateral pressure was assumed to remain constant below 55 ft depth, since the change was negligible.

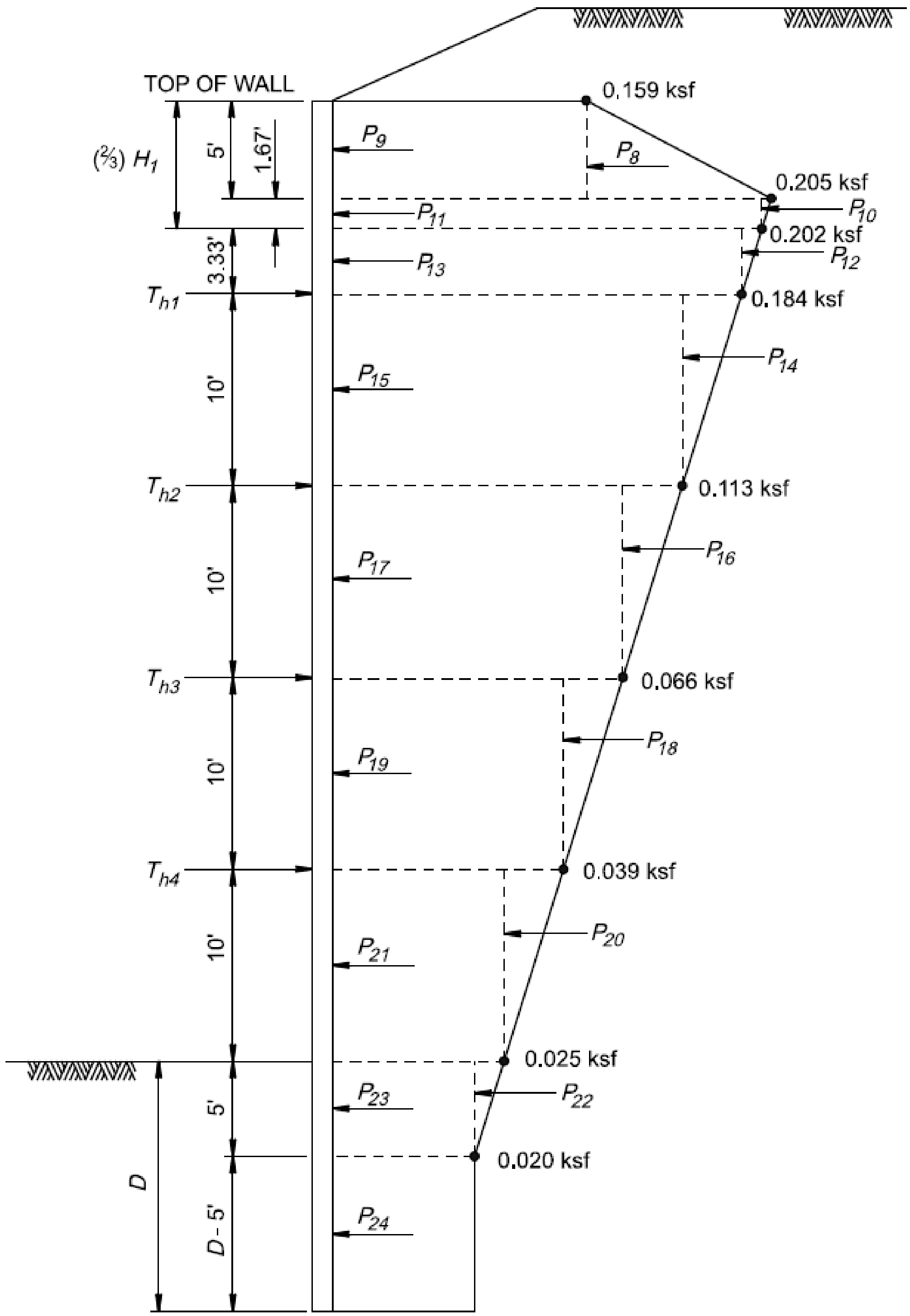


Figure 11.2.4-9 Lateral Pressure Diagram due to *LS* Based on Boussinesq Method for Strength I Limit State

- Calculate factored lateral forces due to *LS* for a single pile

  Factored lateral forces due to *LS* above the excavation line using 8 feet pile spacing:

  $P_8 = 0.5(5)(0.205 - 0.159)(8) = 0.92 \text{ kip}$

  $P_9 = 0.159(5)(8) = 6.36 \text{ kip}$

  $P_{10} = 0.5(1.67)(0.205 - 0.202)(8) = 0.02 \text{ kip}$

  $P_{11} = 0.202(1.67)(8) = 2.70 \text{ kip}$

  $P_{12} = 0.5(3.33)(0.202 - 0.184)(8) = 0.24 \text{ kip}$

  $P_{13} = 0.184(3.33)(8) = 4.90 \text{ kip}$

  $P_{14} = 0.5(10)(0.184 - 0.113)(8) = 2.84 \text{ kip}$

  $P_{15} = 0.113(10)(8) = 9.04 \text{ kip}$

  $P_{16} = 0.5(10)(0.113 - 0.066)(8) = 1.88 \text{ kip}$

  $P_{17} = 0.066(10)(8) = 5.28 \text{ kip}$

  $P_{18} = 0.5(10)(0.066 - 0.039)(8) = 1.08 \text{ kip}$

  $P_{19} = 0.039(10)(8) = 3.12 \text{ kip}$

  $P_{20} = 0.5(10)(0.039 - 0.025)(8) = 0.56 \text{ kip}$

  $P_{21} = 0.025(10)(8) = 2.0 \text{ kip}$

  Factored lateral forces due to *LS* below the excavation line using 2 feet diameter hole:

  $P_{22} = 0.5(5)(0.025 - 0.020)(2) = 0.03 \text{ kip}$

  $P_{23} = 0.020(5)(2) = 0.20 \text{ kip}$

  $P_{24} = 0.020(D - 5)(2) = 0.04D - 0.20 \text{ kip}$

- Determine D by taking moments about T_{hnL} (T_{h4L} in this example) for lower section, per the Modified Hinge method (Refer to Figure 11.2.4-6)

  Factored driving and resisting moments, about T_{h4L}, using Modified Hinge method:

Table 11.2.4-9 Strength I Limit State Factored Driving & Resisting Moments

Driving Force (kip)	Arm (ft)	Driving Moment, M_{DR} (k-ft)
$P_6 = 53.65$	$\frac{3.33}{2}$	89.33
$P_7 = 53.65$	$\frac{6.67}{3} + 3.33$	297.94
$P_{a1} = 5.232D$	$\frac{D}{2} + 10$	$2.616D^2 + 52.32D$
$P_{a2} = 0.052D^2$	$\frac{2D}{3} + 10$	$0.035D^3 + 0.52D^2$
$P_{20} = 0.56$	$\frac{10}{3}$	1.87
$P_{21} = 2.0$	$\frac{10}{2}$	10.0
$P_{22} = 0.03$	$10 + \frac{5}{3}$	0.35
$P_{23} = 0.20$	$10 + \frac{5}{2}$	2.5
$P_{24} = 0.04D - 0.20$	$10 + \frac{5}{2} + \frac{D}{2}$	$0.02D^2 + 0.4D - 2.5$
Resisting Force (kip)	Arm (ft)	Resisting Moment, M_{RS} (k-ft)
$P_p = 1.291D^2$	$\frac{2D}{3} + 10$	$0.861D^3 + 12.91D^2$

$$M_{DR} = 0.035D^3 + 3.156D^2 + 52.72D + 399.49$$

$$M_{RS} = 0.861D^3 + 12.91D^2$$

Pile embedment D required to provide moment equilibrium:

$$M_{RS} = M_{DR}$$

$$-0.826D^3 - 9.754D^2 + 52.72D + 399.49 = 0$$

$$D^3 + 11.809D^2 - 63.826D - 483.64 = 0$$

$D = 7.04$ ft $\rightarrow$ Try 8 ft (Compare with pile embedment D provided by Geotechnical Services in step 1, and use controlling D)

Note: The Hinge method is another acceptable method for calculating pile embedment D, but it does not provide moment equilibrium at the pile tip. In the Hinge method, ground anchor loads need to be calculated before the pile embedment D can be determined, therefore the Hinge method will be demonstrated later in this example problem.

11.2.4.3.4 Anchors (Refer to AASHTO Article 11.9.5.1)

The following calculations are needed as intermediate steps in order to design the ground anchors:

- Calculate factored ground anchor loads, using the Modified Hinge method (Refer to Figure 11.2.4-6)

$T_{h1} =$ Calculated from $\sum M_2 = 0$

$$\begin{aligned}\sum M_2 = {} & P_1\left[\frac{1}{3}(6.67) + 3.33 + 10\right] + P_2\left(\frac{3.33}{2} + 10\right) + P_3\left(\frac{10}{2}\right) + P_8\left(\frac{5}{3} + 1.67 + 3.33 + 10\right) \\ & + P_9\left(\frac{5}{2} + 1.67 + 3.33 + 10\right) + P_{10}\left[\frac{2}{3}(1.67) + 3.33 + 10\right] + P_{11}\left(\frac{1.67}{2} + 3.33 + 10\right) \\ & + P_{12}\left[\frac{2}{3}(3.33) + 10\right] + P_{13}\left(\frac{3.33}{2} + 10\right) + P_{14}\left(\frac{2}{3}\right)(10) + P_{15}\left(\frac{10}{2}\right) - T_{h1}(10) = 0\end{aligned}$$

$$\begin{aligned}\sum M_2 = {} & 53.65\left[\frac{1}{3}(6.67) + 3.33 + 10\right] + 53.65\left(\frac{3.33}{2} + 10\right) + 160.96\left(\frac{10}{2}\right) \\ & + 0.92\left(\frac{5}{3} + 1.67 + 3.33 + 10\right) + 6.36\left(\frac{5}{2} + 1.67 + 3.33 + 10\right) \\ & + 0.02\left[\frac{2}{3}(1.67) + 3.33 + 10\right] + 2.7\left(\frac{1.67}{2} + 3.33 + 10\right) \\ & + 0.24\left[\frac{2}{3}(3.33) + 10\right] + 4.9\left(\frac{3.33}{2} + 10\right) + 2.84\left(\frac{2}{3}\right)(10) + 9.04\left(\frac{10}{2}\right) - T_{h1}(10) = 0\end{aligned}$$

$$T_{h1} = \frac{2{,}554.46}{10} = 255.45 \text{ kip}$$

$$T_1 = \frac{T_{h1}}{\cos(15^\circ)} = \frac{255.45}{\cos(15^\circ)} = 264.46 \text{ kip}$$

$$T_{h2u} = P_1 + P_2 + P_3 + P_8 + P_9 + P_{10} + P_{11} + P_{12} + P_{13} + P_{14} + P_{15} - T_{h1}$$
$$= 53.65 + 53.65 + 160.96 + 0.92 + 6.36 + 0.02 + 2.7 + 0.24 + 4.9 + 2.84 + 9.04 - 255.45$$
$$= 39.83 \text{ kip}$$

$$T_{h2L} = \text{Calculated from} \sum M_3 = 0$$

$$\sum M_3 = P_4\left(\frac{10}{2}\right) + P_{16}\left(\frac{2}{3}\right)(10) + P_{17}\left(\frac{10}{2}\right) - T_{h2L}(10) = 0$$

$$\sum M_3 = 160.96\left(\frac{10}{2}\right) + 1.88\left(\frac{2}{3}\right)(10) + 5.28\left(\frac{10}{2}\right) - T_{h2L}(10) = 0$$

$$T_{h2L} = \frac{843.73}{10} = 84.37 \text{ kip}$$

$$T_{h2} = T_{h2u} + T_{h2L} = 39.83 + 84.37 = 124.20 \text{ kip}$$

$$T_2 = \frac{T_{h2}}{\cos(15°)} = \frac{124.20}{\cos(15°)} = 128.58 \text{ kip}$$

$$T_{h3u} = P_4 + P_{16} + P_{17} - T_{h2L} = 160.96 + 1.88 + 5.28 - 84.37 = 83.75 \text{ kip}$$

$$T_{h3L} = \text{Calculated from} \sum M_4 = 0$$

$$\sum M_4 = P_5\left(\frac{10}{2}\right) + P_{18}\left(\frac{2}{3}\right)(10) + P_{19}\left(\frac{10}{2}\right) - T_{h3L}(10) = 0$$

$$\sum M_4 = 160.96\left(\frac{10}{2}\right) + 1.08\left(\frac{2}{3}\right)(10) + 3.12\left(\frac{10}{2}\right) - T_{h3L}(10) = 0$$

$$T_{h3L} = \frac{827.60}{10} = 82.76 \text{ kip}$$

$$T_{h3} = T_{h3u} + T_{h3L} = 83.75 + 82.76 = 166.51 \text{ kip}$$

$$T_3 = \frac{T_{h3}}{\cos(15°)} = \frac{166.51}{\cos(15°)} = 172.38 \text{ kip}$$

$$T_{h4u} = P_5 + P_{18} + P_{19} - T_{h3L} = 160.96 + 1.08 + 3.12 - 82.76 = 82.40 \text{ kip}$$

$$T_{h4L} = \text{Calculated from} \sum F = 0 \text{ for DEL}$$

$$T_{h4L} = P_6 + P_7 + P_{20} + P_{21} + P_{22} + P_{23} + P_{24} + P_{a1} + P_{a2} - P_p$$
$$= 53.65 + 53.65 + 0.56 + 2.0 + 0.03 + 0.2 + \left[0.04(7.04) - 0.2\right] + (5.232)(7.04)$$
$$+(0.052)(7.04)^2 - (1.291)(7.04)^2 = 85.60 \text{ kip}$$

$$T_{h4} = T_{h4u} + T_{h4L} = 82.40 + 85.60 = 168.0 \text{ kip}$$

$$T_4 = \frac{T_{h4}}{\cos(15°)} = \frac{168.0}{\cos(15°)} = 173.93 \text{ kip}$$

- Determine factored design load (*FDL*) for each ground anchor

$$FDL_n = T_n$$

$$FDL_1 = 264.46 \text{ kip}$$

$$FDL_2 = 128.58 \text{ kip}$$

$$FDL_3 = 172.38 \text{ kip}$$

$$FDL_4 = 173.93 \text{ kip}$$

Note: The designer may change the vertical spacing of the ground anchors to better balance the design loads. The recommended lock off load, *LL*, equals 0.55*FDL*; however, when the movement needs to be minimized, the recommended *LL* equals 0.67*FDL* per Caltrans best practices.

- Design the ground anchors

Ground anchors are designed by the Contractor based on the following criteria:

$$FTL = 1.0FDL$$

$$A_{s(min)} = \frac{1.0FTL}{0.75f_{pu}}$$

where:

FTL = factored test load (kip)

$A_{s(min)}$ = minimum cross sectional area of prestressing steel in ground anchor (in.^{2})

f_{pu} = specified minimum ultimate tensile strength of prestressing steel (ksi)

- Calculate factored ground anchor loads and pile embedment D, using the Hinge method for comparison purposes only (Refer to Figure 11.2.4-10).

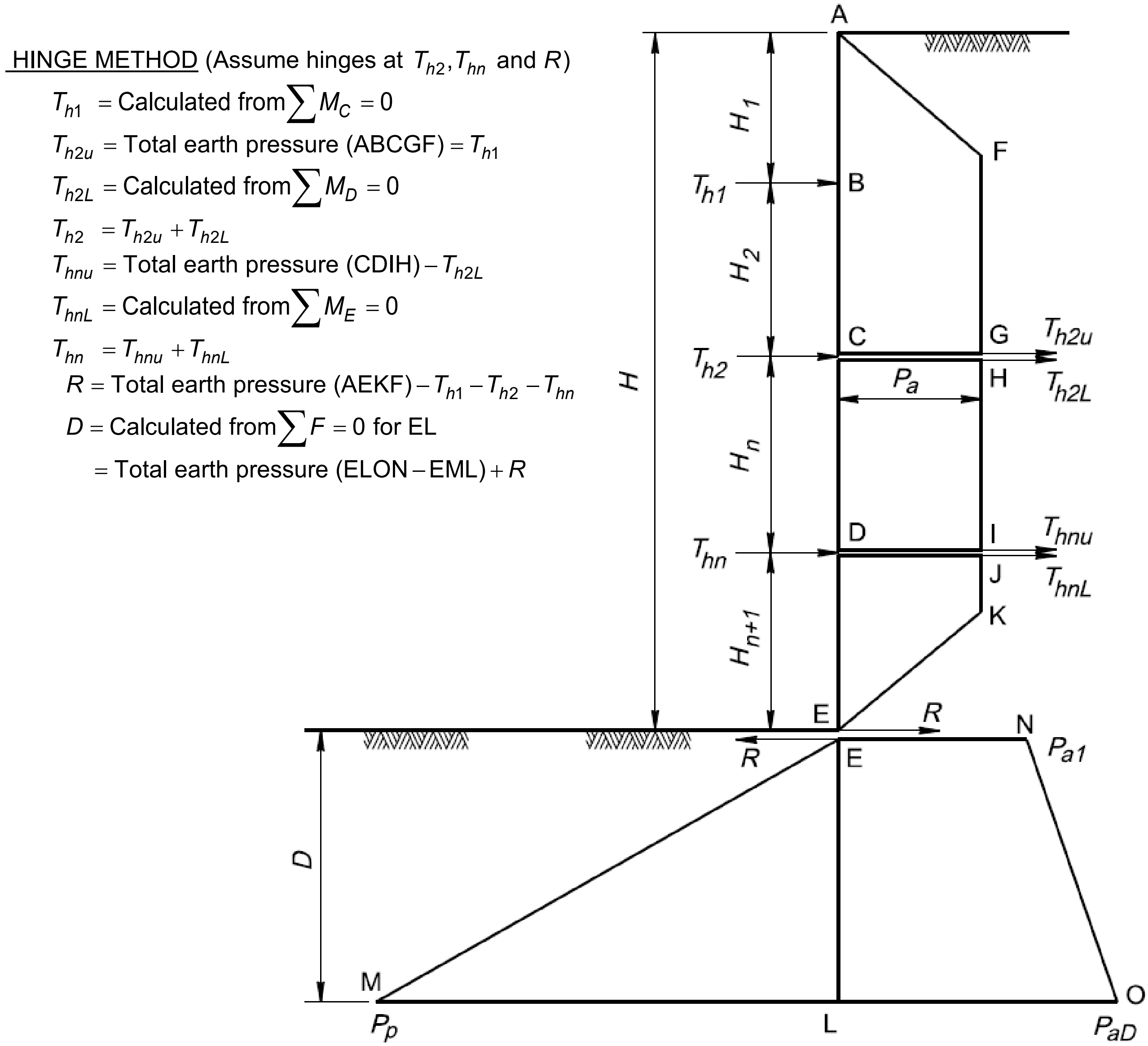


Figure 11.2.4-10 Hinge Method Calculation of Ground Anchor Loads and Pile Embedment for Multilevel Wall (Refer to AASHTO Fig. C11.9.5.1-2)

Factored ground anchor loads, based on the Hinge method:

The Hinge method and Modified Hinge method produce the same ground anchor loads for the top anchors. Only the lowermost ground anchor load differs between the two methods and needs to be recalculated.

$$T_1 = 264.46 \text{ kip}$$

$$T_2 = 128.58 \text{ kip}$$

$$T_3 = 172.38 \text{ kip}$$

$$T_{h4u} = 82.40 \text{ kip}$$

$$T_{h4L} = \text{Calculated from} \sum M_E = 0$$

$$\sum M_E = P_6\left(\frac{3.33}{2}+6.67\right)+P_7\left(\frac{2}{3}\right)(6.67)+P_{20}\left(\frac{2}{3}\right)(10)+P_{21}\left(\frac{10}{2}\right)-T_{h4L}(10)=0$$

$$\sum M_E = 53.65\left(\frac{3.33}{2}+6.67\right)+53.65\left(\frac{2}{3}\right)(6.67)+0.56\left(\frac{2}{3}\right)(10)+2.0\left(\frac{10}{2}\right)-T_{h4L}(10)=0$$

$$T_{h4L} = \frac{699.47}{10} = 69.95 \text{ kip}$$

$$T_{h4} = T_{h4u} + T_{h4L} = 82.40 + 69.95 = 152.35 \text{ kip}$$

$$T_4 = \frac{T_{h4}}{\cos(15^\circ)} = \frac{152.35}{\cos(15^\circ)} = 157.72 \text{ kip} < 173.93 \text{ kip from Modified Hinge method}$$

Pile embedment D, based on the Hinge method (Refer to AASHTO Article C11.9.5.1):

$$R = P_1 + P_2 + P_3 + P_4 + P_5 + P_6 + P_7 + P_8 + P_9 + P_{10} + P_{11} + P_{12} + P_{13} + P_{14}$$
$$+P_{15} + P_{16} + P_{17} + P_{18} + P_{19} + P_{20} + P_{21} - T_{h1} - T_{h2} - T_{h3} - T_{h4}$$

$$R = 53.65 + 53.65 + 160.96 + 160.96 + 160.96 + 53.65 + 53.65 + 0.92$$
$$+6.36 + 0.02 + 2.70 + 0.24 + 4.90 + 2.84 + 9.04 + 1.88 + 5.28 + 1.08 + 3.12$$
$$+0.56 + 2.0 - 255.45 - 124.20 - 166.51 - 152.35 = 738.42 - 698.51 = 39.91 \text{ kip}$$

$$D = \text{Calculated from} \sum F = 0 \text{ for EL}$$

$$P_{22} + P_{23} + P_{24} + P_{a1} + P_{a2} - P_p + R$$
$$= 0.03 + 0.2 + (0.04D - 0.2) + 5.232D + 0.052D^2 - 1.291D^2 + 39.91 = 0$$

$$-1.239D^2 + 5.272D + 39.94 = 0$$

$$D = 8.19 \text{ ft} > 7.04 \text{ ft from Modified Hinge method}$$

Note: Typically, the Hinge method decreases the load on the lowermost anchor, but increases the pile embedment (passive pressure), therefore the Hinge method may be used when base stability is adequate (stable soil below the excavation line).

11.2.4.3.5 Vertical Wall Elements (Refer to AASHTO Article 11.9.5.2)

For Strength I Limit State, try HP14x117 Grade 50 steel beam with E = 29,000 ksi.

The following calculations are needed as intermediate steps in order to check the factored flexural and shear resistances of the piles:

- Calculate the maximum factored moment and shear in the piles

  For simplicity, use CT-Flex to calculate the maximum factored moment and shear. CT-Flex produces the following shear and moment diagrams using the Modified Hinge method:

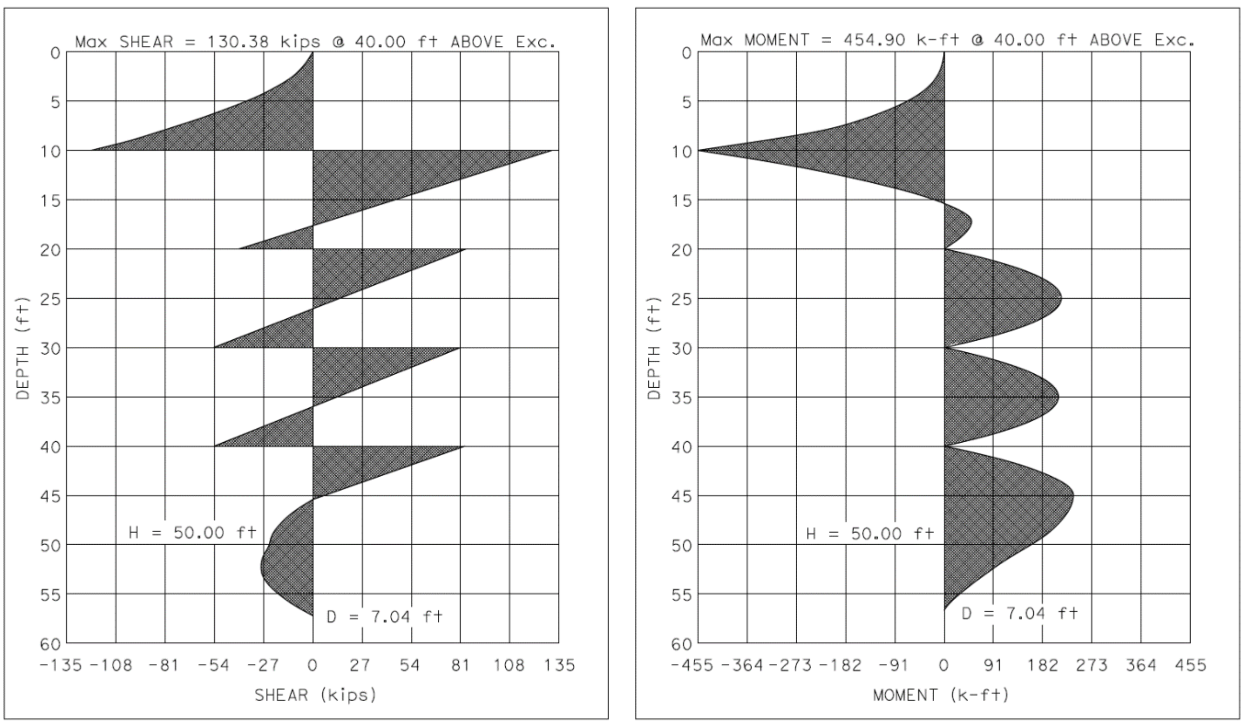


Figure 11.2.4-11 Factored Shear & Moment Diagrams for Strength I Limit State

Note: Both the Hinge and Modified Hinge methods are simplified methods used in CT-Flex to solve an indeterminate problem. The designer may use other software such as CSiBridge for a more refined analysis.

- Check factored flexural resistance of the piles

  Factored flange stress, f_{bu}:

$$f_{bu} = \frac{12M_{max}}{S_x} = \frac{12(454.90)}{172} = 31.74 \text{ ksi}$$

where:

S_x = section modulus = 172 in.^{3} for HP14x117

Factored flexural resistance:

Since the location of the maximum moment is above the excavation line, use the equation for discretely braced flanges.

$$\phi_f F_{nc} \geq f_{bu} + \left(\frac{1}{3}\right) f_l \qquad \text{(AASHTO 6.10.8.1.1-1)}$$

For a compact, nonslender section:

$$F_{nc} \geq R_b R_h F_{yc} \qquad \text{(AASHTO 6.10.8.2.2-1)}$$

By substitution:

$$\phi_f \left(R_b R_h F_{yc}\right) = 0.9(1.0)(1.0)(50) = 45 \text{ ksi} > 31.74 \text{ ksi} \quad \text{OK}$$

where:

ϕ_f = steel resistance factor for flexure = 0.9 (Refer to Table 11.2.4-5)

f_l = flange lateral bending stress = 0 ksi

R_b = web load shedding factor = 1.0 (Refer to AASHTO Article 6.10.1.10.2)

R_h = hybrid factor = 1.0 (Refer to AASHTO Article 6.10.1.10.1)

F_{yc} = specified minimum yield strength of a compression flange = 50 ksi

- Check factored shear resistance of the piles

  Factored shear force, V_u:

$$V_u = V_{max} = 130.38 \text{ kip}$$

Factored shear resistance of unstiffened webs:

$$\phi_v V_n \geq V_u \qquad \text{(AASHTO 6.10.9.1-1)}$$

$$V_n = V_{cr} = CV_p \qquad \text{(AASHTO 6.10.9.2-1)}$$

$$V_p = 0.58F_{yw}Dt_w \qquad \text{(AASHTO 6.10.9.2-2)}$$

By substitution:

$$\phi_v(C0.58F_{yw}Dt_w) = 1.0(1.0)(0.58)(50)(14.21)(0.805) = 331.73 \text{ kip} > 130.38 \text{ kip OK}$$

where:

ϕ_v = steel resistance factor for shear = 1.0 (Refer totable 11.2.4-5)

C = ratio of the shear buckling resistance to the shear yield strength = 1.0 (Refer to AASHTO Eq. 6.10.9.3.2-4)

F_{yw} = specified minimum yield strength of the web = 50 ksi

D = web depth = 14.21 in. for HP14x117

t_w = web thickness = 0.805 in. for HP14x117

Note: In this example, an H14x117 soldier pile would work for Strength I Limit State.

11.2.4.3.6 Facing (Refer to AASHTO Article 11.9.5.3)

For Strength I Limit State, try full sawn 6x12 Douglas Fir-Larch No. 1 lagging.

The following calculations are needed as intermediate steps in order to determine the required bearing length and check the factored flexural and shear resistances of the wood lagging:

- Calculate the total factored uniform load, W_{lag}, for a single lagging at $2/3H_1$ below the top of the wall

  Facing may be designed assuming simple support between elements, with soil arching, f_a (Refer to AASHTO Article C11.8.5.2).

$$f_a = \frac{0.083pL^2}{0.125pL^2} = 0.664$$

$$W_{lag} = f_a\left(p_a + \Delta_{ph}\right)\left(\frac{w_b}{12}\right) = 0.664\left(2.012 + 0.202\right)\left(\frac{12}{12}\right) \approx 1.47 \text{ kip/ft}$$

where:

w_b = bearing width = 12 in. for full sawn 6x12 lagging

- Determine the bearing length required for lagging resistance in compression perpendicular to the grain

Factored reaction, R_u, as shown in Figure 11.2.4-12:

$$R_u = \frac{W_{lag}\left(L_{clear} + \frac{l_b}{12}\right)}{2}$$

Factored resistance in compression perpendicular to the grain:

$$\phi P_n \geq R_u$$

$$P_n = F_{cp} A_b C_b \qquad \text{(AASHTO 8.8.3-1)}$$

$$A_b = l_b w_b$$

By substitution:

$$\phi\left(F_{cp}\, l_b w_b C_b\right) = \frac{W_{lag}\left(L_{clear} + \frac{l_b}{12}\right)}{2}$$

$$l_b = \frac{W_{lag} L_{clear}}{2\phi(F_{cp} w_b C_b) - \frac{W_{lag}}{12}} = \frac{1.47(6.76)}{2\,(0.9)(0.45)(12)(1.0) - \frac{1.47}{12}}$$

$l_b = 1.04$ in. $\rightarrow$ Use 3 in. min, per 'Soldier Pile Wall Lagging Details' XS sheet

where:

l_b = required bearing length (in.)

$$L_{clear} = S - \frac{b_f}{12} = 8 - \frac{14.885}{12} = 6.76\,\text{ft}$$

b_f = flange width of soldier pile = 14.885 in. for HP14x117

ϕ = resistance factor of wood in compression perpendicular to grain = 0.9 (Refer to Table 11.2.4-3)

F_{cp} = adjusted design value of wood in compression perpendicular to grain = 0.45 ksi (Refer to Table 11.2.4-4)

$C_b =$ bearing adjustment factor $= 1.0$ (Refer to AASHTO Article 8.8.3)

Note: Once the bearing length is determined, check the soldier pile flange width for the minimum constructability requirement, $b_{fmin} \le b_f$. With no clipping of the lagging corners $b_{fmin} = max(l_b, 3")(2)(2) + t_w = 3(2)(2) + 0.805 = 12.805$ in. $<$ 14.885 in. OK

- Calculate the lagging span length, L

$$L = L_{clear} + \frac{l_b}{12}$$

$$L = 6.76 + \frac{1.04}{12} = 6.85 \text{ ft}$$

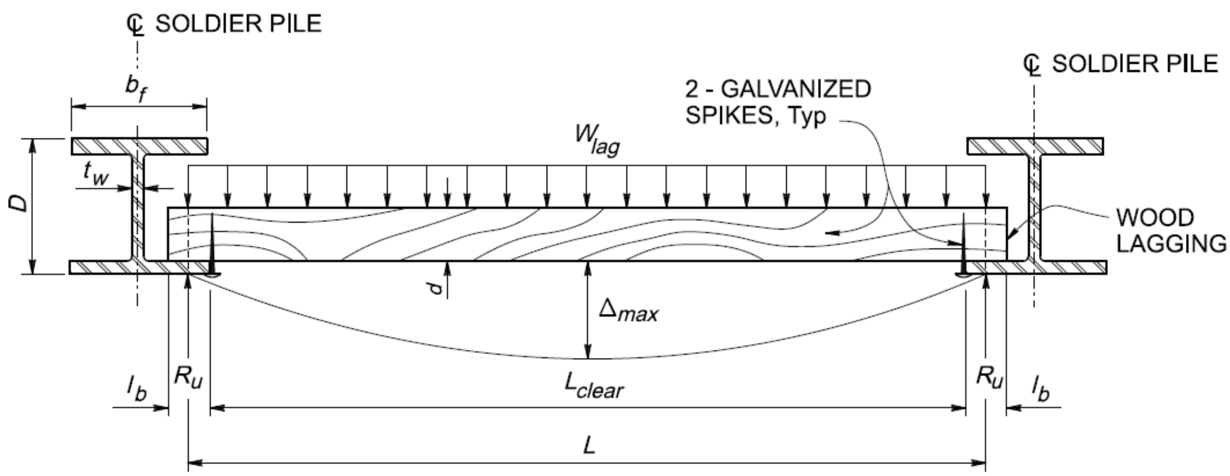


Figure 11.2.4-12 Lagging Parameters

- Check factored flexural resistance of the lagging

Factored moment, M_u:

$$M_u = \frac{W_{lag} L^2}{8} = \frac{1.47(6.85)^2}{8} = 8.62 \text{ k-ft}$$

Factored flexural resistance:

$$\phi M_n \ge M_u$$

$$M_n = \left(F_b S_x C_L\right)/12 \qquad \text{(AASHTO 8.6.2-1)}$$

By substitution:

$$\phi(F_b S_x C_L / 12) = 0.85(1.5)(72)(1.0)/12 = 7.65 \text{ k-ft} < 8.62 \text{ k-ft N.G.}$$

where:

$\phi =$ resistance factor of wood in flexure $= 0.85$ (Refer to Table 11.2.4-3)

$F_b =$ adjusted design value of wood in flexure $= 1.5$ ksi (Refer to Table 11.2.4-4)

$S_x = 72$ in.^{3} for full sawn 6x12 lagging

$C_L =$ stability factor $= 1.0$ (Refer to AASHTO Article 8.6.2)

Since a 6x12 does not have enough flexural resistance in this example, try full sawn 8x12 Douglas Fir-Larch No. 1 lagging with S_x = 128 in.^{3}.

$$\phi(F_b S_x C_L / 12) = 0.85(1.5)(128)(1.0)/12 = 13.6 \text{ k-ft} > 8.62 \text{ k-ft OK}$$

- Check factored shear resistance of the lagging

Factored shear force, V_u, at distance d from the face of support, as defined in AASHTO Article 8.7 (Refer to Figure 11.2.4-12):

$$V_u = W_{lag}\left[\frac{L}{2} - \frac{l_b}{2(12)} - \frac{d}{12}\right] = 1.47 \times \left[\frac{6.85}{2} - \frac{1.04}{2(12)} - \frac{8}{12}\right] = 3.99 \text{ kip}$$

where:

$d =$ lagging depth $= 8$ in. for full sawn 8x12 lagging

Factored shear resistance:

$$\phi V_n \geq V_u$$

$$V_n = F_v w_b d / 1.5 \qquad \text{(Per AASHTO 8.7-2)}$$

By substitution:

$$\phi(F_v w_b d / 1.5) = 0.75(0.14)(12)(8)/1.5 = 6.72 \text{ kip} > 3.99 \text{ kip OK}$$

where:

$\phi =$ resistance factor of wood in shear $= 0.75$ (Refer to Table 11.2.4-3)

$F_v =$ adjusted design value of wood in shear $= 0.14$ ksi (Refer to Table 11.2.4-4)

Note: In this example, an 8x12 wood lagging would work for Strength I Limit State.

11.2.5 MECHANICALLY STABILIZED EMBANKMENT (MSE) DESIGN EXAMPLE

11.2.5.1 Introduction

Mechanically Stabilized Embankment (or Mechanically Stabilized Earth) (MSE) systems behave like a gravity wall, deriving lateral resistance through the self-weight of the reinforced soil mass behind the facing.

MSE systems have three major components:

- Reinforced structure backfill, which carries the bulk of the loading
- Soil reinforcement which augments the fill
- Facing elements which protect the fill

Reinforced structure backfill (also known as reinforced backfill or reinforced soil) refers to the soil material placed within the reinforced soil mass. The retained soil (or retained fill) refers to the material, placed or in situ, directly adjacent to and behind the reinforced backfill zone. The retained soil is the source of lateral earth pressures that the reinforced mass shall resist.

Soil reinforcement is either inextensible (metallic) or extensible (polymeric). The metallic soil reinforcement can be strip reinforcement, bars, or grid type, and the polymeric soil reinforcement may be geotextile or geogrid.

MSE facing can be categorized as stiff, such as concrete blocks, precast concrete panels, or cast-in-place facing, or flexible, such as geogrid, welded wire, and rock filled gabions, etc.

A reinforced soil mass is somewhat analogous to reinforced concrete. In that mass, mechanical properties are improved with reinforcement placed parallel to the principal strain direction to compensate for the soil's lack of tensile resistance. Although there are different types of soil reinforcement, only welded wire mesh is considered in this design example.

Placement of a leveling pad is for the erection of facing elements only, usually 1-foot-wide by 6-inch-thick, and not intended as structural foundation support.

11.2.5.2 General Design Guidelines

MSE shall be designed for external stability of the retaining system as well as internal stability of the reinforced soil mass behind the facing, in accordance with the AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019). Overall and compound stability shall be considered. The structural design of the MSE facing and its connection shall also be considered.

The general design of MSEs includes the following:

- Determine structure dimensions: Design height and front face embedment depth, base width and length of soil reinforcement, and the facing type and its dimensions.
- Determine soil strength parameters for each zone including: Reinforced soil/backfill, foundation soil, and retained soil parameters (Figure 11.2.5-1).
- Calculate MSE loading including: Lateral soil loads, live load surcharge, distributed or point dead loads, earthquake and inertial loads, and water pressure loading.
- Design soil reinforcement: Reinforcement type (metallic strip, wire mesh, geogrid, etc.), configuration, material strength, vertical and horizontal spacing, length and facing connection details.

The applicable Limit States include the following**:**

- Service Limit State: Perform movement and stability analysis at the service limit state for settlement, lateral displacement, and overall and compound stability requirements.
- Strength Limit State: Perform external stability for safety against soil failure, sliding, bearing resistance, and overturning analysis to check limiting eccentricity.
- Strength Limit State: Perform internal stability for safety against structural failure; soil reinforcement pullout, reinforcement strength, and soil reinforcement-facing connection; and facing design.
- Extreme Event Limit State: Seismic design of MSE walls includes internal stability, external stability, facing reinforcement connections, and overall and compound stability.
- Other considerations include: Surface erosion and subsurface drainage, utility openings, obstructions, traffic barriers, etc.

The following requirements apply:

- The minimum soil reinforcement length shall be 70% of the wall height (H). The wall height is measured at the facing from the bottom of the reinforced zone to the top of the reinforced zone or to the finish grade above, as defined in AASHTO Figure 11.10.2-1. In the case of a sloped finish grade, use 70% of the equivalent wall height (H_1) as defined in AASHTO Figure 11.10.6.3.1-1. A minimum 8-foot reinforcement length is applied for uniform compaction and constructability under Caltrans practice. Soil reinforcement length shall be increased as required for a surcharge, external loads, soft foundations, overall (global) stability, etc.
- The soil reinforcement length shall be uniform throughout the entire height of the wall unless substantiating evidence is presented to indicate variation in length gives a satisfactory performance on the individual project (AASHTO 11.10.2.1).
- The minimum front face embedment depth depends on the horizontal slope in front of the wall. Per CA Amendments 11.10.2.2 (Caltrans, 2019), the embedment depth shall not be less than 10% of the design height and not less than 2 feet, and a minimum horizontal bench width of 4 feet shall be provided in front of walls founded on slopes.

- The lowest reinforcement layer shall not be located above the long-term ground surface in front of the wall (AASHTO 11.10.2.2). Hydraulic loads shall be considered when ordinary highwater intercepts the MSE facing or structure backfill (AASHTO 11.10.10.3). MSE should not be used where the floodplain erosion or scour may undermine the reinforced fill zone or facing or supporting footing (AASHTO 11.10.1).
- Seismic detailing should also be addressed for MSE walls in seismically active areas. Table 11.2.5-1 shows the procedure used in the design example of Section 11.2.5.3.

Table 11.2.5-1 shows the procedure used in the design example of Section 11.2.5.3:

Table 11.2.5-1 Summary of Steps in Analysis of MSE

Step	Item
1	Establish project requirement including geometry, construction constraints, and identify all external loads (transient and /or permanent)
2	Establish engineering properties of foundation, retained, and reinforced soils
3	Choose facing type and soil reinforcement type
4	Select resistance factors
5	Establish length of the soil reinforcement and embedment depth of MSE
6	Check external stability, iterate Steps 1 to 4 as necessary to meet all stability criteria
7	Select internal load and resistance factors
8	Establish soil reinforcement configuration, spacing, and capacity
9	Determine internal stability at each reinforcement level
10	Tabulate results and iterate Steps 7 to 9 as necessary to meet all stability criteria
11	Design facing elements, drainage details, etc.

11.2.5.3 Design Example

Figure 11.2.5-1 shows a typical MSE configuration with a level backfill. Per Caltrans practice, the standard MSE design height may be increased by 20 in. to accommodate the traffic barrier attached to a concrete slab floating above the MSE. For simplicity, this example problem assumes no barrier slab above the MSE. The example problem illustrates the analysis of an MSE with a level backfill and live load surcharge. Designer needs to apply all external loads as applicable in the project. Other external loads that ordinarily need to be checked are not included in this example. The extreme event limit state is not considered in this example.

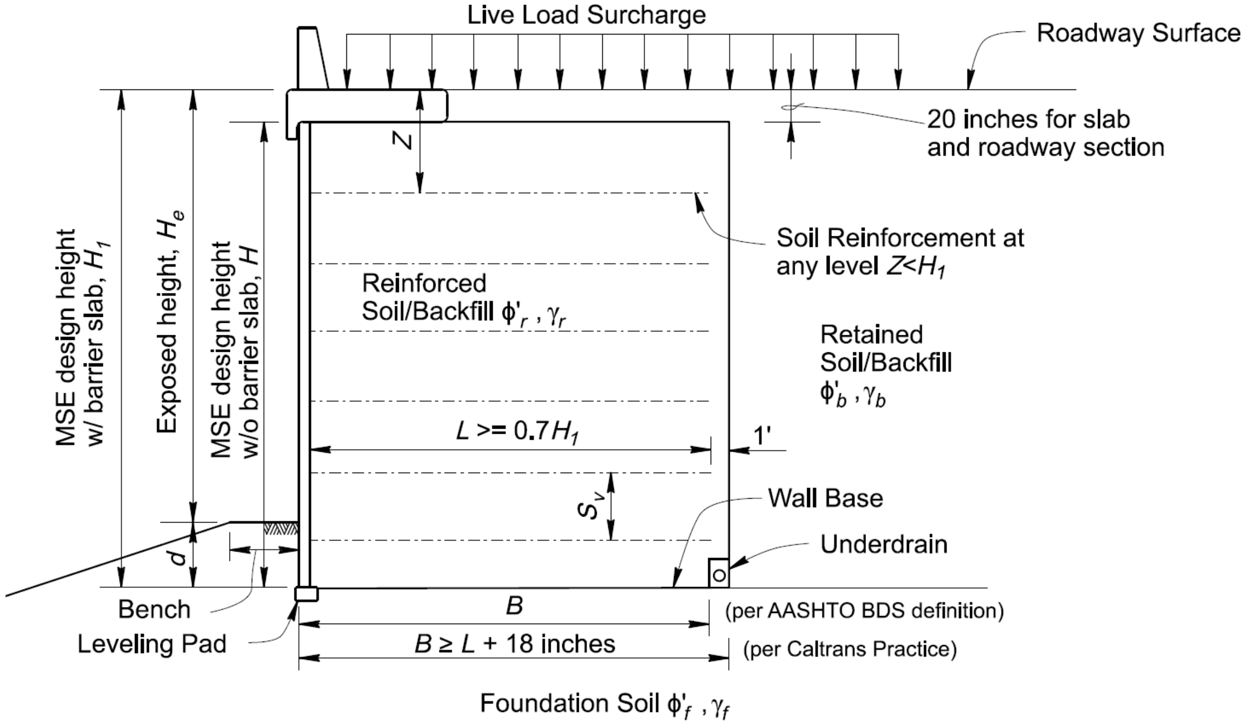


Figure 11.2.5-1 Configuration showing various parameters for analysis of an MSE with level backfill, live load surcharge

11.2.5.3.1 MSE Example Data

The design data for the MSE example are as follows:

- Design separation of grades or exposed height: $H_e = 23\,\text{ft}$
- Design life: 75 years
- Precast facing panel: 5-foot-wide x 5-foot-tall x 0.5-foot-thick with swaged connections for soil reinforcement spaced 30 inches apart. (Note: Caltrans XS sheet details will be used throughout this problem.)
- Type of soil reinforcement: Grade 65 steel welded wire mesh, galvanized with 2 oz./ft^2 zinc coating per AASHTO BDS.

  $f_y = 65\,\text{ksi}$

  Wire sizes = W15 & W20

  Mat width, b = 2.5 ft

The Design Engineer shall consult with the project Geoprofessional for the appropriate soil properties and recommended design parameters. In this design example (for illustration purposes only), assume the following:

- Retained backfill: $\phi_b^{'} = 30^\circ$, $\gamma_b = 0.12\,\text{kcf}$
- Foundation material: $\phi_f^{'} = 30^\circ$, $\gamma_f = 0.12\,\text{kcf}$
- Permissible net contact stress at service limit state assuming 2” settlement, $q_{pn\_ser} = 7.5\,\text{ksf}$
- Factored nominal bearing resistance at strength limit state, $q_{r\_str} = 10.5\,\text{ksf}$

Therefore, in this example: $\phi_f^{'} = \phi_b^{'} = 30^\circ$

Gradation and electrochemical properties of the reinforced zone of structure backfill in this problem will comply with the requirements specified in Caltrans Standard Specification 47-2.02C for metallic soil reinforcement. Caltrans practice assigns typical strength parameters to the specified MSE structure backfill based on Caltrans research, as follows:

- Reinforced zone, MSE structural backfill: $\phi_r^{'} = 34^\circ$, $\gamma_r = 0.12\,\text{kcf}$

- Live Load Surcharge (from AASHTO Table 3.11.6.4-2): $h_{eq} = 2\text{ ft}$.

11.2.5.3.2 Design Requirements

Perform the following design calculations for the MSE in accordance with the AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019).

- Select Depth of Embedment and Length of Soil Reinforcement
- Estimate Unfactored Loads
- Summarize Applicable Load and Resistance Factors
- Design for Service Limit State
- Design for Strength Limit State
- Evaluate Internal Stability

11.2.5.3.3 Select Depth of Embedment and Length of Soil Reinforcement

Caltrans practice applies $d = 0.1H$ (Figure 11.2.5-1), not less than 2 feet, as the minimum embedment depth, which is consistent with AASHTO guidance for a level slope in front of the wall.

$H_e = 23\text{ ft}$,

$H = H_e + d = H_e + 0.1H$,

Therefore, $H = H_e / 0.9 = 25.6\text{ ft}$

The minimum initial length of soil reinforcement is assumed to be $0.7H$ or 8 ft, whichever is greater. This length will be verified as part of the design process. The length of the soil reinforcement is assumed to be constant throughout the height to limit differential settlements across the reinforced zone and avoid overstressing the reinforcements.

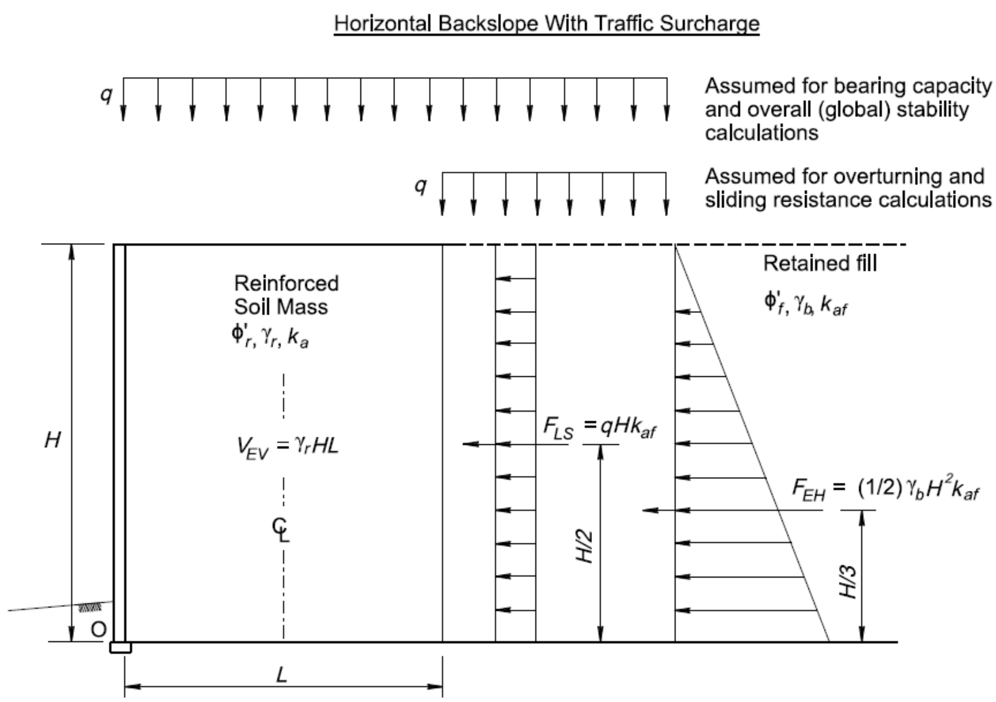
$$L = 0.7H = 17.9 \text{ ft}$$

Which is rounded up to the nearest half a foot for constructability. Thus, for this problem, L = 18 ft.

Note – Caltrans practice adds an additional foot of MSE structure backfill after the length of the soil reinforcement as shown in Figure. This increases the base width, B, of the MSE overall by at least 1 ft behind the reinforced soil and the thickness of the facing in front. All layouts should be based on $B \geq 18 \text{ in.} + L$; however, design calculations are based on L. In practice, other design considerations may cause an increase in these dimensions. For example, increasing embedment depth, d, may increase bearing resistance, or increasing the base width, B, by lengthening L, may improve the overall slope stability.

11.2.5.3.4 Estimate Unfactored Loads

The following equations present the equations for unfactored loads and moment arms about Point O shown in Figure 11.2.5-2. The moments are the product of the respective forces and moment arms. Each force is assigned a designation representing the applicable load type as per CA Amendment to AASHTO Tables 3.4.1-1 and 3.4.1-2.



Note: All moments are about point O; M_{EV}, M_{EH}, M_{V_LS} and M_{F_LS} not shown.

Figure 11.2.5-2 Illustration of forces and moments

To compute the numerical values of various forces and moments, the parameters provided in Section 11.2.5.3.2 are used. Using the values of the various friction angles, the coefficients of lateral earth pressure are as follows:

$$k_a = \frac{\left(1-\sin\left(\phi_r^{'}\right)\right)}{\left(1+\sin\left(\phi_r^{'}\right)\right)} = 0.283$$

$$k_{af} = \frac{\left(1-\sin\left(\phi_f^{'}\right)\right)}{\left(1+\sin\left(\phi_f^{'}\right)\right)} = 0.333$$

or

$$k_a = \tan^2\left(45^\circ - \frac{\phi_r^{'}}{2}\right) = 0.283$$

$$k_{af} = \tan^2\left(45^\circ - \frac{\phi_f^{'}}{2}\right) = 0.333$$

Equations for unfactored vertical forces, *V,* and associated moments, *M*, per unit length of the wall:

$$V_{EV} = \gamma_r HL$$

$$M_{EV} = V_{EV}\left(\frac{L}{2}\right)$$

$$V_{LS} = \gamma_f h_{eq} L$$

$$M_{V\_LS} = V_{LS}\left(\frac{L}{2}\right)$$

Where h_{eq} is the equivalent height of soil as defined in Section11.2.5.3.1, V_{EV} is the vertical force due to reinforced soil mass, M_{EV} is the associated moment about point *O* due to V_{EV}, V_{LS} is the vertical force due to live load surcharge, M_{V_LS} is the associated moment about point *O* due to V_{LS}.

Equations for unfactored horizontal forces, *F,* and associated moment per unit length of the wall:

$$F_{EH} = \frac{1}{2}k_{af}\gamma_b\left(H\right)^2$$

$$M_{EH} = F_{EH}\left(\frac{H}{3}\right)$$

$$F_{LS} = k_{af}\gamma_b h_{eq} H$$

$$M_{F\_LS} = F_{LS}\left(\frac{H}{2}\right)$$

Where F_{EH} is the horizontal force due to retained fill pressure, M_{EH} is the associated moment about point O due to F_{EH}, F_{LS} is the horizontal force due to live load surcharge, M_{F_LS} is the associated moment about point O due to F_{LS}.

Using equations above, numerical values of unfactored forces and moments are calculated below. Refer to Figure 11.2.5-2 for notations of various forces (forces are calculated as per unit length, one foot of the wall length).

Unfactored vertical forces and moments:

$$V_{EV} = (0.12)(25.6)(18) = 55.3\ \frac{\text{kip}}{\text{ft}}$$

$$M_{EV} = (55.3)\left(\frac{18}{2}\right) = 497.7\ \frac{\text{kip-ft}}{\text{ft}}$$

$$V_{LS} = (0.12)(2)(18) = 4.3\ \frac{\text{kip}}{\text{ft}}$$

$$M_{V\_LS} = (4.3)\left(\frac{18}{2}\right) = 38.7\ \frac{\text{kip-ft}}{\text{ft}}$$

Unfactored horizontal forces and moments:

$$F_{EH} = \frac{1}{2}(0.333)(0.12)(25.6)^2 = 13.1\ \frac{\text{kip}}{\text{ft}}$$

$$M_{EH} = (13.1)\left(\frac{25.6}{3}\right) = 111.8\ \frac{\text{kip-ft}}{\text{ft}}$$

$$F_{LS} = (0.333)(0.12)(2)(25.6) = 2.0\ \frac{\text{kip}}{\text{ft}}$$

$$M_{F\_LS} = (2.0)\left(\frac{25.6}{2}\right) = 25.6\ \frac{\text{kip-ft}}{\text{ft}}$$

11.2.5.3.5 Summarize Applicable Load and Resistance Factors

The following summarizes the applicable load factors from AASHTO CA Table 3.4.1-1 and AASHTO CA Table 3.4.5.1-1 for various LRFD load combinations listed in Section 11.2.5.3.4 and the applicable resistance factors for evaluation of resistances from

AASHTO CA 11.5.7-1 (AASHTO 2019).

Applicable Load Factors:

Service I Limit State

$$\gamma_{ser\_EV} = 1.00$$

$$\gamma_{ser\_EH} = 1.00$$

$$\gamma_{ser\_LS} = 1.00$$

Strength Ia Limit State (Bearing Resistance):

$$\gamma_{str\_Ia\_EV} = 1.35$$

$$\gamma_{str\_Ia\_EH} = 1.50$$

$$\gamma_{str\_Ia\_LS} = 1.75$$

Strength Ib Limit State (Sliding and Eccentricity):

$$\gamma_{str\_Ib\_EV} = 1.00$$

$$\gamma_{str\_Ib\_EH} = 1.50$$

$$\gamma_{str\_Ib\_LS} = 1.75$$

Extreme Event I Limit State (Not included in this design example)

Applicable Resistance Factors:

Sliding of MSE on Foundation Soil	$\phi_{sliding} = 1.00$
Bearing Resistance	$\phi_{\text{bearing}} = 0.65$
Tensile Resistance (For Steel Mats)	$\phi_{\text{tensile}} = 0.80$
Pullout Resistance	$\phi_{\text{pullout}} = 0.90$
Flexure and Shear Resistance	$\phi_{\text{Flexure}} = 0.90$

11.2.5.3.6 Design for Service Limit State

11.2.5.3.6.1 Overall Global and Compound Stability

The check for overall global and compound stability falls outside the scope of this design example. This type of analysis is typically performed by a Geoprofessional using a slope analysis program. The Structure Designer should update the Geoprofessional with the latest MSE dimensions whenever a change is made in design.

Typically, overall global and compound stability should be investigated at the following:

- Service I load combination with a resistance factor of 0.65 (AASHTO 11.6.2.3)
- When MSE external dimensions are changed, a recheck of overall global and compound stability may be required.

11.2.5.3.6.2 Settlement Analysis

Settlement is evaluated at Service I limit state. The allowable settlement will be specified by the Structure Designer and the Geoprofessional (design team). The Geoprofessional provides the corresponding permissible net contact stress to the Structure Designer for checking with the net uniform (for soil) or maximum (for rock) bearing stress.

All settlements should be referenced to the top front of the wall. The differential settlement may happen along the wall layout line (LOL) as well as perpendicular to wall LOL along the soil reinforcement. Check the reinforcement connection strength if any differential settlement occurs between the MSE face and the rear end of the reinforced soil mass.

Assuming the permissible net contact stress is provided by the Geoprofessional based on 2-inch settlement:

Vertical load at base of MSE

$$\sum V = \gamma_{\text{ser\_EV}} V_{EV}$$

$$\sum V = (1.00)(55.3) = 55.3\ \frac{\text{kip}}{\text{ft}}$$

Resisting moments at Point *O* on the MSE

$$M_{RO} = \gamma_{\text{ser\_EV}} M_{EV}$$

$$M_{RO} = (1.00)(497.7) = 497.7\ \frac{\text{kip-ft}}{\text{ft}}$$

Overturning moments at Point O on the MSE

$$M_{OO} = \gamma_{\text{ser\_EH}} M_{EH} + \gamma_{\text{ser\_LS}} M_{F\_LS}$$

$$M_{OO} = (1.00)(111.8) + (1.00)(25.6) = 137.4\ \frac{\text{kip-ft}}{\text{ft}}$$

Net Moment at Point O

$$M_O = M_{RO} - M_{OO}$$

$$M_O = (497.7) - (137.4) = 360.3 \ \frac{\text{kip-ft}}{\text{ft}}$$

Location of the resultant from Point O

$$a = \frac{M_O}{\Sigma V}$$

$$a = \frac{360.3}{55.3} = 6.52 \text{ ft}$$

Eccentricity from center of the base

$$e_L = \frac{L}{2} - a$$

$$e_L = \frac{18}{2} - 6.52 = 2.5 \text{ ft}$$

Effective width of base of MSE

$$B' = L - 2e_L$$

$$B' = (18) - 2(2.5) = 13.0 \text{ ft}$$

Calculate net uniform bearing stress including the live load surcharge, *LS*, of 4.3 $\frac{\text{kip}}{\text{ft}}$

$$\sigma_V = \frac{\Sigma V}{L - 2e_L}$$

$$\sigma_V = \frac{55.3}{13.0} = 4.3 \text{ ksf}$$

Note: Compare this bearing stress with permissible net contact stress under service limit provided by the Geoprofessional.

For illustration purposes, the factored bearing resistance q_{pn_ser} for service I limit state assumed in Section 11.2.5.3.1 is used herein. Therefore:

$$q_{pn\_ser} = 7.5 \text{ ksf}$$

Bearing check at the base of MSE, capacity to demand ratio (CDR):

$$Bearing\ CDR = \frac{q_{\text{pn\_ser}}}{\sigma_V}$$

$$\textit{Bearing}\ \mathrm{CDR} = \frac{7.5}{4.3} = 1.7 > 1.0$$

Therefore, design is OK.

11.2.5.3.6 Design for Strength Limit State

11.2.5.3.7.1 Evaluate External Stability of the MSE

The external stability of MSE is a function of the various forces shown in Figure 11.2.5-2.

11.2.5.3.7.2 Sliding Resistance at the Base of the MSE

The purpose is to evaluate the sliding resistance at the base of the MSE. In the computation of sliding resistance, the beneficial contribution of live load surcharge to resisting forces and moments is neglected. Note that sliding resistance is a strength limit state check, and therefore service limit state calculations are not performed.

The soil friction angle ϕ used in sliding evaluation is taken as:

$$\phi = \min\left(\phi_r^{'}, \phi_f^{'}\right) = 30^\circ$$

- Strength Ib Limit State:

  Lateral load on MSE

  $$F_m = \gamma_{\mathrm{str\_Ib\_EH}} F_{EH} + \gamma_{\mathrm{str\_Ib\_LS}} F_{LS}$$

  $$F_m = (1.50)(13.1) + (1.75)(2.0) = 23.2\ \frac{\text{kip}}{\text{ft}}$$

  Vertical load at base of MSE without LL surcharge

  $$V_{base} = \gamma_{\mathrm{str\_Ib\_EV}} V_{EV}$$

  $$V_{base} = (1.00)(55.3) = 55.3\ \frac{\text{kip}}{\text{ft}}$$

  Nominal sliding resistance at base of MSE

  $$V_{Nm} = \tan(\phi) V_{base}$$

  $$V_{Nm} = \tan(30^\circ)(55.3) = 31.9\ \frac{\text{kip}}{\text{ft}}$$

  Factored sliding resistance at base of MSE

$$V_{Fm} = \phi_{sliding} V_{Nm}$$

$$V_{Fm} = (1.00)(31.9) = 31.9 \ \frac{\text{kip}}{\text{ft}} \ \text{(AASHTO 10.6.3.4)}$$

Sliding resistance capacity to demand ratio (CDR):

$$\text{CDR} = \frac{V_{Fm}}{F_m}$$

$$\text{CDR} = \frac{31.9}{23.2} = 1.38 > 1.0$$

Therefore, design is OK.

11.2.5.3.7.3 Limiting Eccentricity at the Base of the MSE

The purpose is to evaluate the limiting eccentricity at the base of the MSE. In the computation of limiting eccentricity, the beneficial contribution of live load to resisting forces and moments is neglected. Note that limiting eccentricity is a strength limit state check; therefore, service limit state calculations are not performed. Per AASHTO 11.6.3.3, for a foundation on soil, the location of the resultant of reaction forces shall be within middle two-third of the base width.

- Strength Ib Limit State:

Total vertical load at base of MSE without live load surcharge, *LS*

$$V_{base} = \gamma_{\text{str\_Ib\_EV}} V_{EV}$$

$$V_{base} = (1.00)(55.3) = 55.3 \ \frac{\text{kip}}{\text{ft}}$$

Resisting moments about Point *O* without *LS*

$$M_{RO} = \gamma_{\text{str\_Ib\_EV}} M_{EV}$$

$$M_{RO} = (1.00)(497.7) = 497.7 \ \frac{\text{kip-ft}}{\text{ft}}$$

Overturning moments about Point O

$$M_{OO} = \gamma_{\text{str\_Ib\_EH}} M_{EH} + \gamma_{\text{str\_Ib\_LS}} M_{F\_LS}$$

$$M_{OO} = (1.50)(111.8) + (1.75)(25.6) = 212.5 \ \frac{\text{kip-ft}}{\text{ft}}$$

Net Moment about Point O

$$M_O = M_{RO} - M_{OO}$$

$$M_O = (497.7) - (212.5) = 285.2\ \frac{\text{kip-ft}}{\text{ft}}$$

Location of the resultant force on base of MSE from Point O

$$a = \frac{M_O}{V_{base}}$$

$$a = \frac{285.2}{55.3} = 5.2\ \text{ft}$$

Eccentricity at base of MSE

$$e_L = \frac{L}{2} - a$$

$$e_L = \frac{18}{2} - 5.2 = 3.8\ \text{ft}$$

Limiting eccentricity for strength limit state

$$e = \frac{L}{3}$$

$$e = \frac{18}{3} = 6\ \text{ft}$$

$$e > e_L$$

Therefore, design is OK.

Effective width of base of MSE

$$B' = B - 2e_L$$

For simplicity here (see note below), take:

$$B' = L - 2e_L$$

$$B' = (18) - 2(3.8) = 10.4\ \text{ft}$$

Note: For simplicity, MSE base width, *B*, is taken as *L* in this design example, instead of *B* = *L* + facing thickness (+ 1 foot per Caltrans' definition). For relatively thick facing elements, it may be reasonable to include the facing dimensions and weight in bearing calculation (i.e. use B in lieu of L as shown in AASHTO Figure 11.10.2-1 [C11.10.5.4, AASHTO 2017]). However, this is not as conservative as using *L*, especially if the contractor will be allowed to use a proprietary product on the project. Any change proposed in the facing may impact a calculation based on *B* during construction of the project.

11.2.5.3.7.4 Bearing Resistance at the Base of the MSE

For bearing resistance check, the effect of live load is included in bearing demand computations since it creates larger bearing stresses. The bearing stress at the base of the MSE can be computed as follows:

$$\sigma_V = \frac{\Sigma V}{L - 2e_L}$$

Where $\Sigma V = V_{EV} + V_{LS}$ is the resultant of vertical forces and the load eccentricity, e_L, is calculated by principles of statics using appropriate loads and moments with the applicable load factors.

In LRFD, factored Nominal bearing stress σ_V is compared with the factored bearing resistance computed for strength limit state. The Service I load combination is evaluated to compute the bearing stress.

- Strength Ia Limit State

  Vertical load at base of MSE including *LS* on top

  $$\Sigma V = \gamma_{\text{str\_Ia\_EV}} V_{EV} + \gamma_{\text{str\_Ia\_LS}} V_{LS}$$

  $$= (1.35)(55.3) + (1.75)(4.3) = 82.2\ \frac{\text{kip}}{\text{ft}}$$

  Resisting moments at Point O on the MSE

  $$M_{RO} = \gamma_{\text{str\_Ia\_EV}} M_{EV} + \gamma_{\text{str\_Ia\_LS}} M_{V_{LS}}$$
  $$= (1.35)(497.7) + (1.75)(38.7) = 739.6\ \frac{\text{kip-ft}}{\text{ft}}$$

  Overturning moments at Point O

  $$M_{OO} = \gamma_{\text{str\_Ia\_EH}} M_{EH} + \gamma_{str\_Ia\_LS} M_{F\_LS}$$
  $$= (1.50)(111.8) + (1.75)(25.6) = 212.5\ \frac{\text{kip-ft}}{\text{ft}}$$

Net Moment at Point O

$$M_O = M_{RO} - M_{OO} = (739.6) - (212.5) = 527.1 \frac{\text{kip-ft}}{\text{ft}}$$

Location of the resultant from Point O

$$a = \frac{M_O}{\Sigma V} = \frac{527.1}{82.22} = 6.4 \text{ ft}$$

Eccentricity from center of wall base

$$e_L = \frac{L}{2} - a = \frac{18}{2} - 6.4 = 2.6 \text{ ft}$$

Effective width of base of MSE

$$B' = L - 2e_L = (18) - 2(2.6) = 12.8 \text{ ft}$$

Calculate bearing stress based on the uniform distribution:

$$\sigma_V = \frac{\Sigma V}{L - 2e_L} = 82.2/12.8 = 6.4 \text{ ksf}$$

Note: Compare this bearing vertical stress (bearing demand) with factored nominal bearing resistance provided by the Geoprofessional or provide the computed bearing stress and B' to the Geoprofessional for factored nominal bearing resistance evaluation.

For illustration purposes, the factored nominal bearing resistance q_{r_str} for strength limit states assumed in Section 11.2.5.3.1 is used herein. Therefore:

$$q_{r\_str} = 10.5 \text{ ksf}$$

Check bearing resistance exceeds the loads at base of the MSE:

$$q_{\text{r\_str}} > \sigma_V \text{ ?}$$

$$\text{Bearing CDR} = \frac{q_{\text{r\_str}}}{\sigma_V}$$

$$\text{Bearing CDR} = \frac{10.5}{6.4} = 1.6 > 1.0$$

Therefore, design is OK.

11.2.5.3.8 Evaluate Internal Stability

Internal stability analysis is calculated using the Simplified Method or the Coherent Gravity Method. The Simplified Method is applied to both steel and geosynthetic reinforced wall systems. The Coherent Gravity Method is applied primarily to steel soil reinforcement systems.

The design of the soil reinforcement requires the following steps for each level of soil reinforcement:

1. Establish the soil reinforcement layout and configuration
2. Compute the horizontal stress, σ_H , at each reinforcement level
3. Determine the maximum load, T_{max}, as the demand in the soil at each reinforcement level
4. Calculate the length of soil reinforcement in the resistant zone to establish the pullout resistance
5. Determine CDR of pullout resistance
6. Establish tensile capacity of the longitudinal soil reinforcement;
7. Determine CDR of soil reinforcement strength
8. Establish facing connection strength
9. Determine CDR of facing connection
10. Summarize the soil reinforcement pattern for each level with corresponding CDR’s.

11.2.5.3.8.1 Establish Layout of the Soil Reinforcement

Typically, the soil reinforcement layout is directly related to the facing elements selected for use. For this example, the chosen 5-foot by 5-foot precast panel accommodates a single steel mat centered from side to side. This creates a horizontal reinforcement spacing (S_h) of 5 feet on center. Selecting a soil reinforcement vertical spacing (S_v) of 2.5 feet will place two levels of steel mat reinforcement per panel. However, the center of the soil layer behind the panel center, to leave sufficient clear distance all around for connection development lengths within the concrete panel. This places the soil reinforcement 1.25 feet from the edge of the panel top and bottom. This also ensures enough soil surrounding each soil reinforcement mat for the strength properties assumed in this analysis.

Facing panels are laid out from bottom up on the project site, with every layer of soil reinforcement at the same elevation from end to end of the MSE facing for improved constructability. To accommodate elevation variations in the grade, steps are included in the bottom panel layout, but without changing the soil reinforcement elevations or their connection lengths in the panels. Caltrans practice encourages steps that accommodate a panel or a half panel in height. This is due to the panel placement in vertical running bond layout. The vertical spacing at the second layer from the top of the panel columns are locally adjusted as necessary to fit changes in the grade at the top of the wall (see Caltrans' XS sheets for Mechanically Stabilized Embankment) with a minimum of 6 inches of soil between layers of soil reinforcement. The topmost layer of soil reinforcement is attached parallel with the top of the panel at a fixed distance throughout the wall to always be at the same clear distance from the finished grade for roadway maintenance.

However, the elevation of the topmost layer of soil reinforcement shall be adjusted to accommodate the barrier slab and layered materials of the roadway section. The topmost layer can be uniformly placed 3 ft from the finished grade. This prevents soil reinforcement from encroaching into the road section, keeping it clear of the barrier slab for future maintenance. However, the barrier slab is not considered in this example.

To place the soil reinforcement layers along the wall height, the following steps should be considered:

The bottom soil reinforcement layer is always placed at 1.25 ft above the leveling pad. The top soil reinforcement layer is always set at 1′-2″ from the top of the top panel. As shown in Figure 11.2.5-7, the spacing between the top two layers of soil reinforcement will vary depend on the wall profile. In this example, assuming the S_v = 2.5 ft and the height of 25.6 ft, the variable distance is 0.68 ft, and there are 11 layers of soil reinforcement.

When there is a barrier slab above the MSE, the top soil reinforcement layer will be 15 inches below the bottom of the barrier slab. Using the definition of depth *Z,* as shown in Figure 11.2.5-1, the preliminary vertical layout of the soil reinforcement is chosen, as shown in Table 11.2.5-2 below, with a total of 11 levels of soil reinforcement.

Table 11.2.5-2 Summary of Soil Reinforcement Layout

<table>
<tr><th>Reinforcement Layer #</th><th>Zi from FG at top (ft)</th><th>Delta Zi (ft)</th><th>Half of Delta Zi (ft)</th><th>Sv (ft)</th></tr>
<tr><td rowspan="2">11 (Top)</td><td rowspan="2">1.17</td><td>1.17</td><td>1.17</td><td rowspan="2">1.51</td></tr>
<tr><td rowspan="2">0.68</td><td>0.34</td></tr>
<tr><td rowspan="2">10</td><td rowspan="2">1.85</td><td>0.34</td><td rowspan="2">1.59</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">9</td><td rowspan="2">4.35</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">8</td><td rowspan="2">6.85</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">7</td><td rowspan="2">9.35</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">6</td><td rowspan="2">11.85</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">5</td><td rowspan="2">14.35</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">4</td><td rowspan="2">16.85</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">3</td><td rowspan="2">19.35</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">2</td><td rowspan="2">21.85</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td rowspan="2">2.5</td><td>1.25</td></tr>
<tr><td rowspan="2">1 (Bottom)</td><td rowspan="2">24.35</td><td>1.25</td><td rowspan="2">2.5</td></tr>
<tr><td>1.25</td><td>1.25</td></tr>
</table>

Where Delta Z_i is the difference in elevation of two adjacent layers, then the soil reinforcement is placed within each layer of backfill defined by S_v.

In the following sections, the detailed calculations will be illustrated for layer (or level) 7.

11.2.5.3.8.2 Establish Soil Reinforcement Configuration

The spacing of transverse wires, S_t, is varied depending on the level of soil reinforcement to optimize the design from an economical perspective. In this design example, assume the layout of longitudinal and transverse wires as shown in Table 11.2.5-3 below.

Table 11.2.5-3 Assumed Bar Mat Configuration for Internal Stability Analysis

Layer #	Longitudinal Wire	Transverse Wire	Spacing of Transverse Wires, S_t (in.)
9 to 11	W15	W15	6
7 to 8	W15	W15	12
4 to 6	W20	W15	18
1 to 3	W20	W15	24

The resistance provided by galvanized steel mat soil reinforcement is based on the design life and estimated loss of steel over the design life due to corrosion.

The nominal diameters, D, of W15 and W20 wires are as follows:

For W15 wires: $D_{W15} = 0.437$ in.

For W20 wires: $D_{W20} = 0.505$ in.

Assume the reinforced backfill conforms to criteria defined in Caltrans Standard Specifications 47-2.02C. Per AASHTO CA 11.10.6.4.2a, the basis for calculating the loss in thickness due to corrosion for this reinforced backfill type:

Loss of galvanizing = 10 years

Carbon Steel Loss = 1.1 mil per year per side after zinc depletion

As per section 11.2.5.3.1, the design life is 75 years. The base carbon steel will lose thickness for 75 years - 10 years = 65 years at a rate of 1.1 mil/year/side. Therefore, the anticipated diameter and area after 75 years for W15 and W20 can be calculated as follows:

- For W15 Wires

$$D_{75} = 0.437\text{ in.} - \frac{(1.1\text{ mil / year / side})(65\text{ years})(2\text{ sides})}{1000} = 0.437\text{ in.} - 0.143\text{ in.} = 0.294\text{ in.}$$

Based on a 0.294 in. diameter wire, the cross-sectional area of W15 at the end of 75 years will be equal to:

$$D_{75\_W15} = 0.294\text{ in.}$$

$$A_{75\_W15} = \frac{\pi D_{75\_W15}^2}{4} = 0.0679\text{ in.}^2\text{ per wire}$$

- For W20 Wires

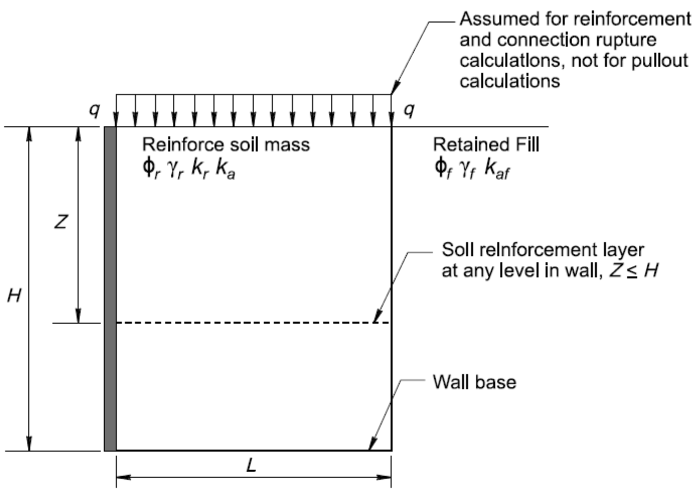
$$D_{75} = 0.505\text{ in.} - \frac{(1.1\text{mil / year / side})(65\text{ years})(2\text{ sides})}{1000} = 0.505\text{ in.} - 0.143\text{ in.} = 0.362\text{ in.}$$

Based on a 0.362 in. diameter wire, the cross-sectional area of W20 at the end of 75 years will be equal to:

$$D_{75\_W20} = 0.362\text{ in.}$$

$$A_{75\_W20} = \frac{\pi D_{75\_W20}^2}{4} = 0.1029\text{ in.}^2\text{ per wire}$$

11.2.5.3.8.3 Calculate Maximum Load at Each Reinforcement Level



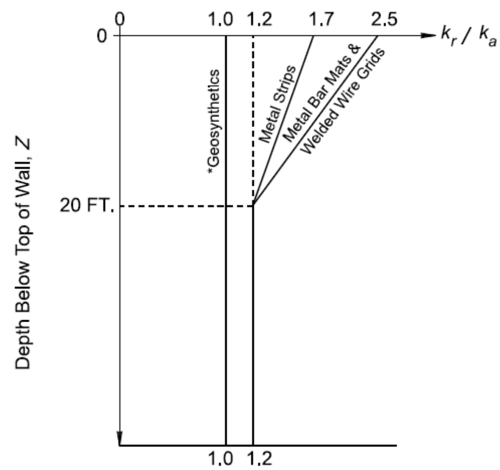


Figure 11.2.5-3 Left: Calculation of vertical stress for horizontal backfill condition; Right: Variation of lateral stress ratio k_r/k_a with depth.

The horizontal stress, σ_H, at any depth within the MSE is based on the soil load and any applicable external loads as summarized below (AASHTO C11.10.6.2.1). However, external loads are not included in this example:

$\sigma_H = \sigma_{H\_EH} + \sigma_{H\_LS}$ for checking connection and soil reinforcement strength

$\sigma_H = \sigma_{H\_EH}$ for checking pullout

$$\sigma_{H\_EV} = \gamma_{p\_EV} k_r \sigma_{v\_EV} = \gamma_{p\_EV} k_r \gamma_r Z$$

$$\sigma_{H\_LS} = \gamma_{p\_LS} k_r q = \gamma_{p\_LS} k_r \gamma_r h_{eq}$$

Once the horizontal stress is computed at any given layer of soil reinforcement, the maximum load to apply to each level of soil reinforcement, T_{max}, is computed. The T_{max} should be calculated twice for internal stability. First to check soil reinforcement and connection rupture with LL surcharge and second to check pullout without LL surcharge.

T_{max} is computed as follows:

$$T_{max} = \sigma_H S_v$$

where S_v is the vertical spacing for each level of soil reinforcement as calculated above.

The computations for T_{max} are illustrated at $Z = 9.37\text{ ft}$ which is Layer 7 in the assumed vertical layout of soil reinforcement. Use:

$$\gamma_{p\_EV} = \gamma_{str\_\mathrm{Ia}\_EV}$$

$$\gamma_{p\_LS} = \gamma_{str\_\mathrm{Ia}\_LS}$$

Obtain k_r by linear interpolation. At Z=0, k_r equals 2.5k_a, and at Z=20, k_r equals 1.2k_a. Thus, at Z=9.37 it equals 1.89k_a. As $k_a = 0.283$, then $k_r = (1.89)(0.283) = 0.535$ at Z = 9.37 ft

$$\sigma_H = \sigma_{H\_EH} + \sigma_{H\_LS} = \gamma_{p\_EV} k_r \gamma_r Z + \gamma_{p\_LS} k_r \gamma_r h_{eq}$$

$$\sigma_H = (1.35)(0.535)(0.12)(9.37) + (1.75)(0.535)(0.12)(2) = 1.04\text{ ksf}$$

From Table 11.2.5-2, the vertical spacing at Layer 7 is: $S_{vt} = 2.5\text{ ft}$

The maximum load at Layer 7 is computed as:

$T_{max} = \sigma_H S_v = 2.60\text{ kip/ft}$ for checking connection and soil reinforcement strength

Repeat the above procedure without including the live load surcharge:

$T_{max} = 2.03\text{ kip/ft}$ for checking pullout strength

Using similar computations, the various quantities can be developed at each layer of soil reinforcement and load combinations.

11.2.5.3.8.4 Check the Pullout Resistance of Soil Reinforcement

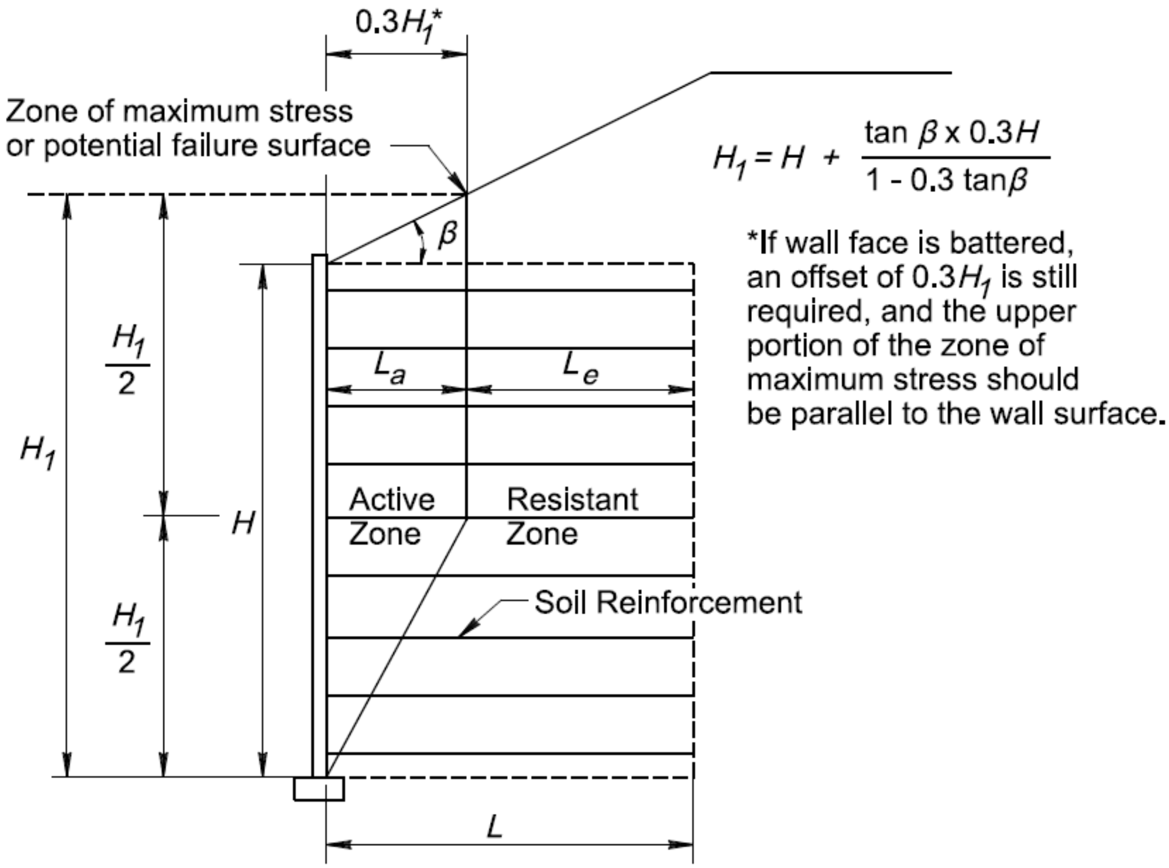


Figure 11.2.5-4 Location of potential failure surface for internal stability design of MSE walls (inextensible soil reinforcement)

The pullout resistance develops after the stress transfer takes place between the structure backfill and the soil reinforcement. The reinforcement pullout resistance shall be checked at each level against pullout failure.

As shown in Figure 11.2.5-4, only the length of the soil reinforcement behind the zone of maximum stress can be used in pullout resistance. In this example, $\beta = 0$, so H_1 reduces to H.

Compute effective length L_e as follows:

Since $Z < H/2$

$$L_e = L - 0.3H = 10.3 \text{ ft}$$

The ultimate pullout resistance (unfactored) per unit of soil reinforcement width (width measured transversely), P_r, of galvanized steel mat soil reinforcement is based on various parameters in the following equation:

$$P_r = (F^*)\alpha(\sigma_{v\_soil})(C)(L_e) \quad \text{(AASHTO C11.10.6.3.2)}$$

In the above equation, the contribution of live load is not included. The steel mat soil reinforcement is considered inextensible with:

$$\alpha = 1 \quad \text{(AASHTO Table 11.10.6.3.2-1)}$$

where α is the scale effect correction factor.

C is the geometry factor, and for steel grids, $C = 2$.

The spacing of transverse wires, S_t, is varied depending on the level of reinforcement to optimize the design from an economical perspective. As shown in Figure 11.2.5-5, the F^* parameter is based on the value of t/S_t which varies from $20(t/S_t)$ at $z = 0$ ft, to $10(t/S_t)$ at $z \geq 20$ ft.

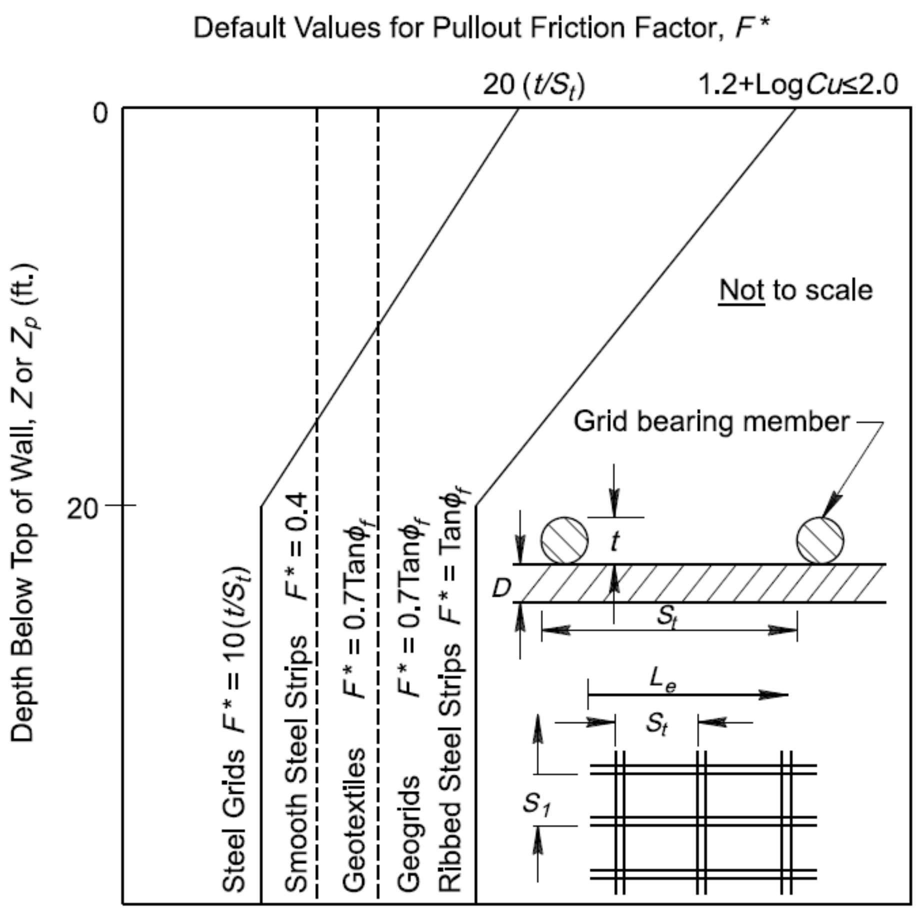


Figure 11.2.5-5 Default values for the pullout friction factor, *F**

For this design example, W15 transverse wires are used for all transverse wires on the mats irrespective of the spacing. Thus, F^* is determined as follows:

For W15 wires, $t = 0.294$ inch after corrosion

When:

$$S_t = 6 \text{ in.} \quad \frac{t}{S_t} = 0.0490 \quad F^*_{z=0} = 0.98 \quad \text{and} \quad F^*_{z=20} = 0.49$$

$$S_t = 12 \text{ in.} \quad \frac{t}{S_t} = 0.0245 \quad F^*_{z=0} = 0.490 \quad \text{and} \quad F^*_{z=20} = 0.245$$

$$S_t = 18 \text{ in.} \quad \frac{t}{S_t} = 0.0163 \quad F^*_{z=0} = 0.326 \quad \text{and} \quad F^*_{z=20} = 0.163$$

$$S_t = 24 \text{ in.} \quad \frac{t}{S_t} = 0.0123 \quad F^*_{z=0} = 0.246 \quad \text{and} \quad F^*_{z=20} = 0.123$$

The computations for P_r are illustrated at $Z = 9.37$ ft which is Level 7, and Z is measured from top of the wall.

Obtain F^* at $Z = 9.37$ ft by linear interpolation between 0.490 at $Z = 0$ and 0.245 at $Z = 20$ ft as follows:

$$F^* = 0.490 + \frac{(9.37 \text{ ft} - 0 \text{ ft})(0.245 - 0.490)}{20 \text{ ft}} = 0.375 \text{ at } Z = 9.37 \text{ ft}$$

For σ_{v_soil}, per AASHTO 11.10.6.3.2, use unfactored vertical stress for pullout resistance, which is:

$$\sigma_{v\_soil} = \sigma_{v\_EV} = (0.120 \text{ kcf})(9.37 \text{ ft}) = 1.124 \text{ ksf}$$

Using σ_{v_soil} at this level as determined above, compute unfactored pullout resistance as follows:

$$P_r = (F^*)\alpha(\sigma_{v\_soil})(C)(L_e)$$

$$P_r = (0.375)1(1.124)(2)(10.3) = 8.68 \text{ kip/ft}$$

Past practice directs a soil reinforcement coverage ratio, R_c, between 1.0 and 0.5 to reduce deformations and provide some redundancy. This example uses $R_c = 0.5$.

Follow AASHTO 11.10.6.3.2-1 and rearrange. Check for:

$$\phi\, P_r R_c \geq T_{max}$$

For pullout resistance, $\phi = 0.9$

$$\phi\, P_r R_c = (0.9)(8.68)(0.5) = 3.91 \text{ kip/ft} \geq T_{max} = 2.03 \text{ kip/ft}$$

Thus, the pullout resistance CDR is:

$$CDR = \frac{\phi\, P_r R_c}{T_{max}} = \frac{3.91}{2.03} = 1.93$$

Using similar computations, the various quantities can be developed at all other levels of soil reinforcement.

11.2.5.3.8.5 Check Factored Long-Term Tensile Resistance of the Soil Reinforcement

The reinforcement strength shall be checked at each level of soil reinforcement, both at zone of maximum stress (see Figure 11.2.5-4), and at the connection of reinforcement to the panel face.

With panel width w_p = 5 ft and coverage ratio R_c = 0.5, the width of reinforcement unit can be calculated as $b = w_p R_c = 2.5$ ft. For the unit width of reinforcement, select a mat with four, Grade 65, W15 longitudinal wires at level 7 for the tensile strength check.

- Rupture at the Zone of Maximum Stress

$$T_{max} \leq \phi T_{al} R_c$$

where T_{max} is calculated in Section 11.2.5.3.8.3, T_{al} is the nominal long-term reinforcement design strength, and is calculated as:

$$T_{al} = \frac{A_c F_y}{b}$$

where A_c is the total area of longitudinal reinforcement corrected for corrosion loss within width of reinforcement unit b, and is calculated as:

$$A_c = (\text{No. of longitudinal bars})\left(A_{75\_W15}\right) = (4)(0.0679) = 0.272 \text{ in.}^2$$

Therefore:

$$T_{al} = \frac{(0.272)(65)}{2.5} = 7.07 \text{ kip/ft}$$

Using the resistance factor $\phi_{tensile} = 0.8$ as listed in Section11.2.5.3.5, the factored tensile resistance is calculated is:

$$\phi T_{al} R_c = (0.8)(7.06)(0.5) = 2.83 \text{ kip/ft} \geq T_{max} = 2.60 \text{ kip/ft}$$

The rupture *CDR* at the zone of maximum stress is:

$$CDR = \frac{2.83}{2.60} = 1.09 \geq 1$$

- Rupture at the Connection to the Facing Panel

$$T_o \leq \phi T_{ac} R_c$$

where T_o is the applied factored load at reinforcement/facing connection (in this example there is no external load), and is equal to T_{max} according to AASHTO BDS 11.10.6.2.2, T_{ac} is nominal long-term reinforcement/facing connection design strength and is calculated similarly as T_{al}.

Caltrans' XS sheet facing panel uses W25 wire embedded in the panel concrete to connect to longitudinal soil reinforcement through a coupler near the panel face.

$$D_{75\_W25} = 0.564 \text{ in.} - \frac{(1.1\text{mil / year / side})(65 \text{ years})(2 \text{ sides})}{1000} = 0.421 \text{ in.}$$

$$A_{75\_W25} = 0.139 \text{ in.}^2$$

$$T_{ac} = \frac{(4)(0.139)(65)}{2.5} = 14.456 \text{ kip/ft}$$

The tensile capacity of the swaged connection depicted on the Caltrans XS sheets may be taken as 95% of the capacity of the wire size shown. In this example there are no external loads on the facing to consider, so set $T_o = T_{max}$, which is consistent with AASHTO BDS 11.10.6.2.2 too. Therefore:

$$\phi T_{ac} R_c = (\phi T_{ac} R_c) = 5.77 \text{ kip/ft}$$

$$0.95(\phi T_{ac} R_c) = (0.95)(\phi T_{ac} R_c) = 5.49 \text{ kip/ft} \geq T_{max} = 2.60 \text{ kip/ft}$$

The CDR at the connection to the facing panel is:

$$\text{CDR} = \frac{5.49}{2.60} = 2.11 \geq 1$$

11.2.5.3.8.6 Tabulate the Results

The computations in Sections 11.2.5.3.8.3 to 11.2.5.3.8.5 are repeated for each layer of soil reinforcement and summarized below in Table 11.2.5-4. Table 11.2.5-4 is generated using a spreadsheet per design steps and formula illustrated in section 11.2.5.3.8.3 to 11.2.5.3.8.5. Soil reinforcement layout configuration is indicated in section11.2.5.3.8.1.

For convenience, convert the design checks into ratios for reporting in the table. Setting the factored loads, T_{max}, as the demand, the ratio is then reported as capacity to demand (CDR) for each design check. This is consistent with available software such as MSEW, as demonstrated in the FHWA GEC (FHWA, 2009a and 2009b).

Table 11.2.5-4 Summary of Soil Reinforcement Calculations

Soil Reinf. Layer #	*Z*, ft	T_{max}, w/ *LS*, (kip/ft)	T_{max}, w/o *LS*, (kip/ft)	Transverse Wire	Longitudinal Wire Size	# of Longitudinal Wires	CDR against Pullout	CDR against Rupture	CDR Connection
11 (Top)	1.17	0.63	0.20	W15 @ 6	W15	4	6.33	4.48	8.71
10	1.85	0.77	0.32	W15 @ 6	W15	4	6.00	3.66	7.12
9	4.35	1.76	1.11	W15 @ 6	W15	4	3.83	1.60	3.11
8	6.85	2.22	1.61	W15 @ 12	W15	4	1.92	1.27	2.47
7	9.35	2.59	2.03	W15 @ 12	W15	4	1.93	1.09	2.11
6	11.85	2.86	2.35	W15 @ 18	W20	4	1.29	1.49	1.92
5	14.35	3.04	2.58	W15 @ 18	W20	4	1.42	1.41	1.80
4	16.85	3.13	2.71	W15 @ 18	W20	4	1.62	1.37	1.75
3	19.35	3.12	2.76	W15 @ 24	W20	4	1.37	1.37	1.76
2	21.85	3.36	3.01	W15 @ 24	W20	4	1.52	1.27	1.63
1 (Bottom)	24.35	3.71	3.35	W15 @ 24	W20	4	1.66	1.15	1.48

11.2.5.4 Design of Facing Elements

The precast facing elements shall be designed as structural elements with appropriate connection strength. In this design example, Caltrans XS sheets pre-approved 5-foot by 5-foot panel system is used here with a slight adjustment about the top wire mat location of top panels.

In general, MSE wall facing shall consider durability, flexibility, strength, compatibility, and adequate anchorage. The factored tensile force in connection should be less than the reduced factored tensile resistance corresponding to the specific limit state.

11.2.5.4.1 Design Parameters for Precast Concrete Facing Panels (Caltrans xs13-020-2)

Design: AASHTO-CA BDS-08 (AASHTO, 2017; Caltrans, 2019)

Live Load $\text{Surcharge} = 240\ \dfrac{\text{lb}}{\text{ft}^2}$

Soil Parameters:

Internal Design: $\phi = 34°,\ \gamma = 0.12\ \text{kcf}$

External Design: $\phi(\text{Retained Backfill}) = 30°,\ \gamma = 0.12\ \text{kcf}$

$\phi(\text{Foundation}) = 30°$

$K_h = 0.2$

Precast Concrete Panels

$f_c^{'} = 4{,}000\ \text{psi}$

$f_y = 60{,}000\ \text{psi}$

Soil Reinforcement:

Welded Wire Mats: $f_y = 65{,}000\ \text{psi}$ (Yield Strength)

Coupler: $f_y = 36{,}000\ \text{psi}$ (Yield Strength)

$\text{Corrosion Rate} = 1.1\ \text{mils/year}$

Reinforced Concrete:

$f_c^{'} = 3{,}600\ \text{psi}$,

$f_y = 60{,}000\ \text{psi}$

Panel Design (Caltrans xs13-020-1)

Typical panel facing design and detail are shown in Figure 11.2.5-6 and Figure 11.2.5-7. Refer to Caltrans XS sheets for details not shown here.

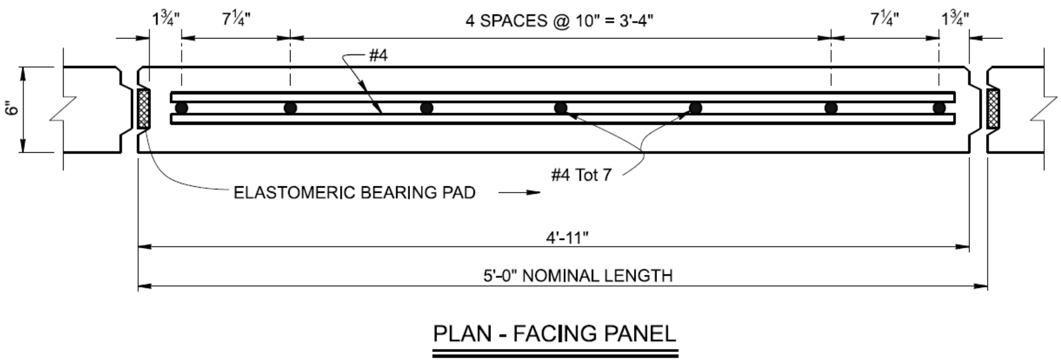


Figure 11.2.5-6 Plan View – Panel Facing Design (not to scale)

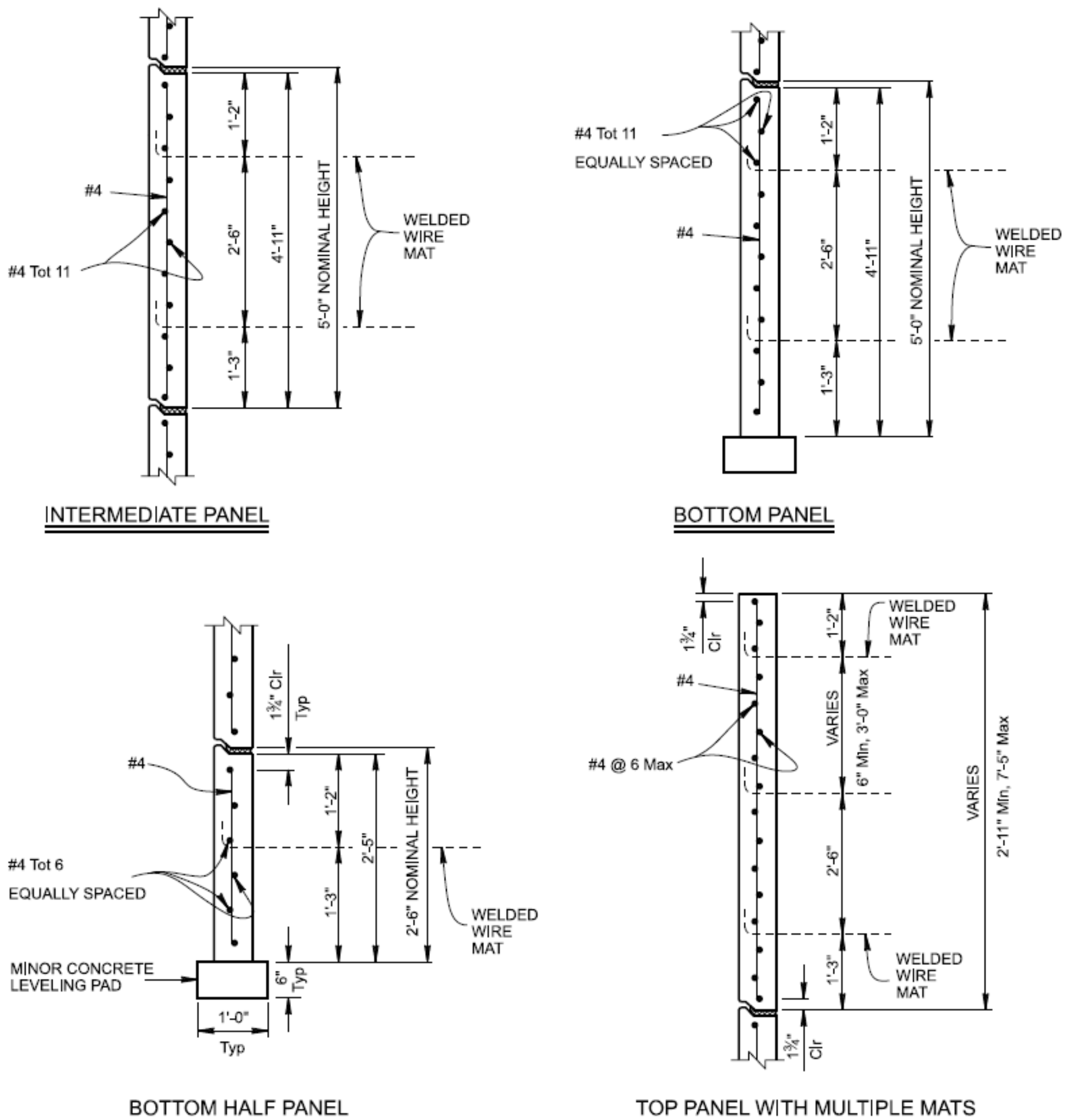


Figure 11.2.5-7 Elevation View – Panel Facing Design (not to scale)

Panel facing design shall follow AASHTO 11.10.2.3, which states the facing element shall be designed to resist the horizontal force in the soil reinforcement at the facing. Per AASHTO 11.10.6.2.2, that force, T_o, is set equal to the maximum factored reinforcement tension, $T_{max,}$ where there is no external load. The designer shall consider all external loads for calculating the concrete facing panels and their connections. Tension may be assumed to be resisted by a uniformly distributed earth pressure on the back of the facing. According to Table 11.2.5-4, layer 1 soil reinforcement at the bottom has the highest tension, T_{max}, demand.

The Caltrans XS facing panel shown in Figure 11.2.5-6 and Figure 11.2.5-7 is checked against this tension demand.

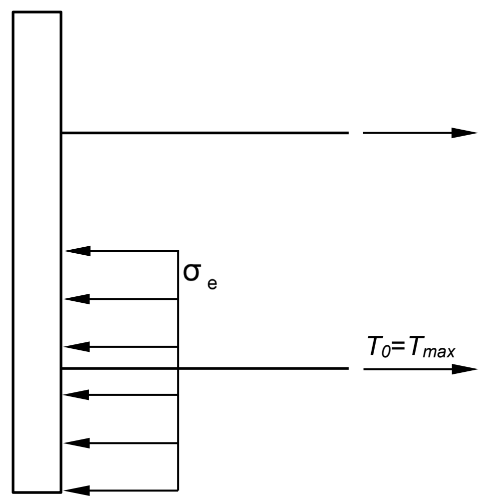


Figure 11.2.5-8 Load for facing design, including reinforcement tension and earth pressure

Figure 11.2.5-8 isolates the bottom panel with a loading diagram shown for the lower portion of the panel (level 1 tributary area):

$$T_o = T_{max} = 3.71 \text{ kip/ft}$$

The equivalent uniform earth pressure per 1-foot width is:

$$\sigma_e = T_o / S_{vt} = (3.71 \text{ kip/ft}) / (2.5 \text{ ft}) = 1.48 \text{ kip/ft/ft}$$

Notice T_{max} is a factored load, therefore the factored maximum moment at the facing is:

$$M_u = \frac{1}{2}\sigma_e L^2 = \frac{1}{2}\sigma_e \left(\frac{1}{2}S_{vt}\right)^2 = 1.16 \text{ kip-ft/ft}$$

Figure 11.2.5-6 shows #4 @ 10, with #4 placed at the center of the panel:

$$A_s = \left(0.20 \text{ in.}^2\right) / \left(10 \text{ in.} / \left(12 \text{in./ft}\right)\right) = 0.24 \text{ in.}^2/\text{ft}$$

The nominal moment capacity of panel per unit width can be calculated:

$$M_n = 3.4 \text{ kip-ft/ft}$$

Check factored moment resistance against demand:

$$M_r = \Phi M_n = (0.9)(3.4 \text{ kip-ft/ft}) = 3.1 \text{kip-ft/ft} > M_u = 1.16 \text{ kip-ft/ft}$$

The panel has sufficient flexural strength. Similarly, the panel shear strength can be checked at critical section in accordance with AASHTO.

The factored maximum shear force at the facing can be calculated as:

$$V_u = \sigma_e L = \sigma_e \left(\frac{1}{2} S_{vt}\right) = 1.85 \text{ kip/ft}$$

The nominal shear capacity of the panel per unit width is calculated (consider concrete shear strength only AASHTO 5.7.3.3-3):

$$V_n = 0.0316 \beta \sqrt{f_c'} b_v d_v = (0.0316)(2)\sqrt{4 \text{kip}}(12 \text{in.})(2.82 \text{in.}) = 4.3 \text{ kip/ft}$$

Check factored moment resistance against demand:

$$V_r = \Phi V_n = (0.9)(4.3 \text{ kip/f}t) = 3.9 \text{ kip/ft} > V_u = 1.86 \text{ kip/ft}$$

The panel has sufficient shear strength.

11.2.5.4.2 Panel Connection Design (Caltrans xs13-020-2)

Figures 11.2.5-9 and 11.2.5-10 show the soil reinforcement connections to the facing panel. The strength of the coupler connector and the embedment of wire in the panel shall be checked against the active force, T_o. Refer to AASHTO 11.10.6.4.4 for details.

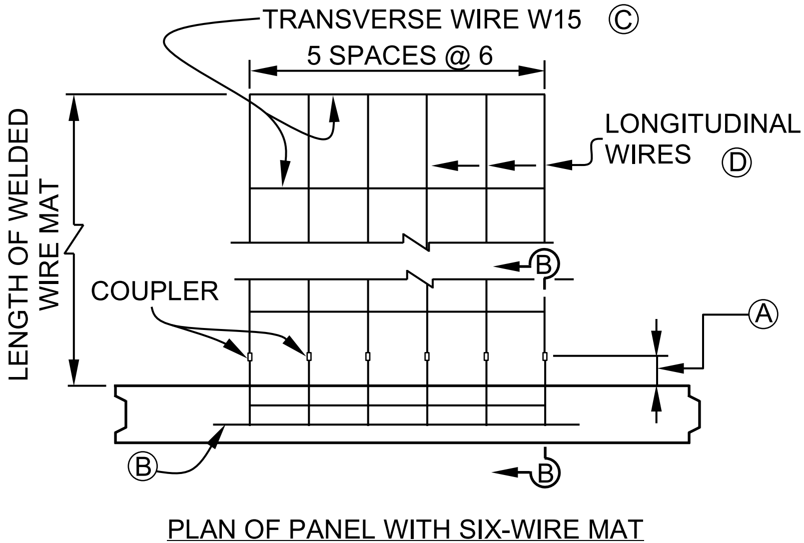


Figure 11.2.5-9 Plan View of Panel Design with Six-Wire Mat (not to scale)

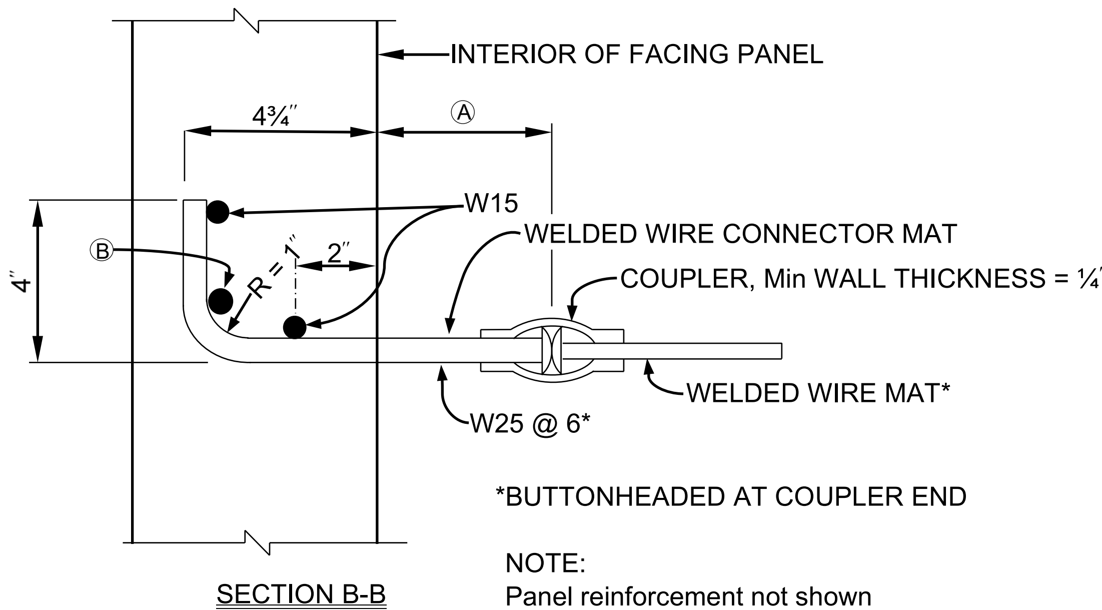


Figure 11.2.5-10 Panel Connection Design, Section B-B of Figure 11.2.5-9 (not to scale)

Notes:

Ⓐ Distance as required to permit coupler to be swaged

Ⓑ Place #4 x 3′-2″, centered on connector mat, but not welded to it

Ⓒ All transverse wires size W15 at various spacing as shown in Table 11.2.5-4

Ⓓ Size of longitudinal wires shown in Table 11.2.5-4

11.2.5.5 Drainage

A standard underdrain, as depicted on Caltrans Bridge Standard Detail Sheets (xs13-020-6), is used for this example. The wall drainage system for an actual project is designed by referencing the standard design and details shown on Caltrans XS sheet and is modified as needed to accommodate specific project requirements.

Outlets and cleanouts should be called out on design plans to be used for construction and as-built use by maintenance in the future.

NOTATION

a	=	distance of resultant force from front base of the MSE wall (ft)
A_{75}	=	cross sectional area of the wire at the end of 75 years design life (in.^{2})
A_c	=	total area of longitudinal reinforcement corrected for corrosion loss (in.^{2})
A_s	=	cross sectional area of reinforcement in facing element per unit width (in.^{2}/ft)
b	=	gross width of reinforcement steel grid/mat (ft); drilled hole diameter (ft)
B	=	Wall base width (ft)
B'	=	effective width of base of MSE wall (ft); effective spread footing width (ft)
c	=	soil cohesion (psf)
C	=	overall reinforcement surface area geometry factor; ratio of the shear buckling resistance to the shear yield strength
C_b	=	bearing adjustment factor
CDR	=	capacity to demand ratio
$\mathrm{C_L}$	=	stability factor
d	=	design embedment depth of vertical element (ft)
d	=	retaining wall displacement in design earthquake (in.)
D	=	nominal diameter of the wire (in.); pile web depth (in.); design embedment depth of vertical element (ft)
D_0	=	embedment depth of vertical element (ft)
D_{75}	=	diameter of the wire at the end of 75 years design life (in.)
dl	=	incremental length along the pile
e	=	limiting eccentricity of the MSE (ft); calculated eccentricity for reinforced concrete wall (ft)
e_L	=	eccentricity due to overturning effect from the center of wall base (ft)
e_r	=	permissible eccentricity for reinforced concrete retaining wall (ft)
F^*	=	reinforcement pullout friction factor
f_a	=	soil arching factor
F_b	=	adjusted design value of wood in flexure (ksi)
f_c'	=	concrete compressive strength at 28 days (ksi)
F_{cp}	=	adjusted design value of wood in compression perpendicular to grain (ksi)
F_{EH}	=	unfactored horizontal force due to earth pressure per unit wall length (kip/ft)
F_{LS}	=	unfactored horizontal force due to surcharge per unit wall length (kip/ft)
F_m	=	factored lateral load on MSE wall per unit wall length (kip/ft)

f_y	=	yield strength of steel reinforcement (ksi)
F_{yf}	=	specified minimum yield strength of the flange (ksi)
F_{yw}	=	specified minimum yield strength of the web (ksi)
g	=	gravity acceleration on earth (ft/sec^2)
H	=	MSE wall height (ft)
H_1	=	equivalent MSE wall height (ft)
H_e	=	exposed MSE wall height (ft)
h_{eq}	=	equivalent height of live load surcharge (ft)
$HPGA$	=	site adjusted horizontal peak ground acceleration at zero period (g)
k_a	=	coefficient of active lateral earth pressure
k_{af}	=	coefficient of active lateral earth pressure in retained fill behind an MSE
k_h	=	horizontal seismic acceleration coefficient
k_{h0}	=	site adjusted horizontal peak ground acceleration coefficient at zero period
k_p	=	coefficient of passive lateral earth pressure
k_r	=	horizontal earth pressure coefficient of reinforced soil/backfill
k_v	=	retaining wall vertical seismic acceleration coefficient
k_y	=	retaining wall horizontal seismic yield acceleration
L	=	length of soil reinforcement (ft)
L_a	=	length of reinforcement in active zone (ft)
l_b	=	required bearing length (in.)
L_{clear}	=	clear span length between flanges (ft)
L_e	=	length of reinforcement in resistant zone (ft)
M	=	moment at any point (kip-ft)
M_{EH}	=	unfactored moment due to earth pressure per unit wall length (kip-ft/ft)
M_{EV}	=	unfactored moment due to earth weight per unit wall length (kip-ft/ft)
M_{F_LS}	=	unfactored moment due to surcharge per unit wall length (kip-ft/ft)
M_n	=	nominal moment capacity of the facing element (kip-ft/ft)
M_O	=	net moment at front base of the MSE wall (kip-ft/ft)
M_{OO}	=	overturning moment at front base of the MSE wall (kip-ft/ft)
M_r	=	factored moment resistance of the facing element (kip-ft/ft)
M_{RO}	=	resisting moment at front base of the MSE wall (kip-ft/ft)
M_u	=	the maximum moment at the facing element (kip-ft/ft)

M_{V_LS} = unfactored moment due to live load surcharge per unit wall length (kip-ft/ft)

p = pressure (ksf);

P = pressure at any point (ksi); lateral load (kip)

P_A = active lateral earth load (kip)

P_{AE} = seismic active lateral earth load (kip)

PGV = site adjusted peak ground velocity (ft/s)

P_P = passive lateral earth load (kip)

P_{PE} = seismic passive lateral earth load (kip)

P_r = ultimate pullout resistance per unit of reinforcement width (kip/ft)

q_{pn_ser} = Permissible net contact stress for foundation soil at service limit (ksf)

q_{r_str} = factored nominal bearing resistance of foundation soil at strength limit (ksf)

R_c = reinforcement coverage ratio

R_h = hybrid factor

S = pile spacing (ft)

S_h = horizontal reinforcement spacing (in.)

S_t = spacing between transverse grid elements (in.)

S_v = vertical reinforcements spacing (ft)

t = wire diameter of soil reinforcement (in.)

T_{ac} = the nominal long-term reinforcement/facing connection design strength (kip)

T_{al} = the nominal long-term reinforcement design strength (kip)

T_{max} = the maximum tension at each level of soil reinforcement per panel width (kip)

T_o = factored tensile load at reinforcement/facing connection per panel (kip/ft)

t_w = web thickness (in.)

V = shear force at any point (kip)

V_{base} = factored vertical load at base of MSE wall per unit wall length (kip/ft)

V_{EV} = unfactored vertical force due to earth weight per unit wall length (kip/ft)

V_{Fm} = factored sliding resistance at base of MSE wall per unit wall length (kip/ft)

V_{LS} = unfactored vertical force due to live load surcharge per unit wall length (kip/ft)

V_n = nominal shear capacity of the panel per unit width (kip/ft)

V_{Nm} = nominal sliding resistance at base of MSE wall per unit wall length (kip/ft)

V_r = factored shear resistance of the panel per unit width (kip/ft)

V_s = soil shear wave velocity (ft/sec)

V_u = factored shear force of the panel per unit width (kip/ft)

w_b = lagging bearing width (in.)

w_p = panel width of precast facing element (ft)

y = deflection at any point

Z = depth from the top of the MSE wall to soil reinforcement (ft)

α = scale effect correction factor

β = inclination of ground slope behind face of wall (degrees)

γ_p = active load factor

θ = slope at any point

σ_H = horizontal earth pressure (ksf)

φ_{ep} = resistance factor for passive soil resistance

φ_r = resistance factor for sliding resistance between soil and footing (dim)

ϕ = resistance factor

ϕ_f = steel resistance factor for flexure=0.9 (Refer to Table 11.2.3.1.1-5)

ϕ_v = steel resistance factor for shear=1.0

δ = angle of lateral earth load (degrees); wall friction angle (degrees)

ϕ_b' = effective internal friction angle of retained soil/backfill (degrees)

ϕ_f = soil friction angle (degrees)

ϕ_f' = effective internal friction angle of foundation soil (degrees)

ϕ_r' = effective internal friction angle of reinforced soil/backfill (degrees)

γ_b = unit weight of retained soil/backfill (kcf)

γ_c = unit weight of reinforced concrete (pcf)

γ_{EQ} = load factor for live load applied simultaneously with seismic loads

γ_f = unit weight of foundation soil (kcf)

γ_r = unit weight of reinforced soil/backfill (kcf)

γ_s = unit weight of soil backfill (pcf, kcf)

γ_{ser} = load factor for service limit state

γ_{str_Ia} = load factor for strength *Ia* limit state

γ_{str_Ib} = load factor for strength *Ib* limit state

σ	=	shear stress on an infinitesimal soil element (psf)
σ_e	=	equivalent uniform earth pressure per unit width of wall (kip/ft/ft)
σ_v	=	bearing stress at base of MSE wall (ksf)
τ	=	normal stress on an infinitesimal soil element (psf)

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