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3.62.1 General

AASHTO 5.1

Retaining wall shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effect, live load and collision forces, and earthquake loads in accordance with the general principles of AASHTO Section 5 and the general principles specified in this section.

1.1 Wall Type Selection

AASHTO 5.2.1

Selection of wall type shall be based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, potential for earthquake loading, presence of deleterious environmental factors, wall site cross-sectional geometry, proximity of physical constraints, tolerable and differential settlement, facing appearance, and ease and cost of construction.

The following wall types are the most commonly used in MoDOT projects

- Mechanically Stabilized Earth Retaining Walls
- Cast-In-Place Concrete Cantilever Retaining Walls
  - Cantilever Walls on Spread Footings
  - Cantilever Wall on Pile Footings
- L-Shaped Walls on Spread Footings

Mechanically Stabilized Earth (MSE) Retaining Walls

AASHTO 5.2.1.4 & 5.8

MSE retaining walls use precast block or panel like facing elements combined with either metallic or geosynthetic tensile reinforcements in the soil mass. MSE walls are preferred over cast-in-place walls because they are usually more economical. Other advantages include a wide variety of design styles, ease and speed of installation, and their ability to accommodate total and differential settlements. Wall design heights upwards of 80 feet are technically feasible (FHFW-SA-96-071). MSE walls may be used to retain fill for end bents of bridge structures.

Situations exist where the use of MSE walls is either limited or not recommended. Some obstacles such as drop inlets, sign truss pedestals or footings, and fence posts may be placed within the reinforcing strip area, however, these obstacles increase the difficulty and expense of providing sufficient reinforcing strips for stability. Box culverts and highway drainage pipes may run through MSE walls, but it is preferable not to run the pipes close to or parallel to the walls. Utilities other than highway drainage should not be constructed within the reinforcing strip area. Be cautious when using MSE walls in a flood plain. A flood could cause scouring around the reinforcement and seepage of the backfill material. Soil reinforcements should not be used where exposure to ground water contaminated by acid mine drainage or other industrial pollutants as indicated by a low pH and high chlorides and sulfates exist. Galvanized metallic reinforcements shall not be used where stray electrical ground currents could occur as would be present near an electrical substation.
Sufficient right-of-way is required to install the reinforcing strips which extend into the backfill area at least 8 feet, 70% of the wall height or as per design requirements, whichever is greater. Finally, barrier curbs constructed over or in line with the front face of the wall shall have adequate room provided laterally between the back of the wall facing and the curb or slab so that load is not directly transmitted to the top wall facing units.

**Concrete Cantilever Wall on Spread Footing**
Concrete cantilever walls derive their capacity through combinations of dead weight and structural resistance. These walls are constructed of reinforced concrete.

Concrete cantilever walls are used when MSE walls are not a viable option. Cantilever walls can reduce the rock cut required and can also provide solutions when there are right of way restrictions. Concrete walls also provide better structural capacity when barrier curbs on top of the walls are required.

Counterforts are used on rare occasions. Sign-board type retaining walls are a special case of counterfort retaining walls. They are used where the soil conditions are such that the footings must be placed well below the finished ground line. For these situations the wall is discontinued 12 inches below the ground line or below the frost line. Counterforts may also be a cost-savings option when the wall height approaches 20 feet (Foundation Analysis and Design by Joseph E. Bowles, 4th ed., 1988). However, other factors such as poor soil conditions, slope of the retained soil, wall length and uniformity in wall height should also be considered before using counterforts.

**Concrete Cantilever Wall on Pile Footing**
Concrete cantilever walls on pile footings are used when the soil conditions do not permit the use of spread footings. These walls are also used when an end bent requires wings longer than 22 feet. In these cases a stub wing is left attached to the end bent and the rest of the wing is detached to become a retaining wall.

**Concrete L-Shaped Retaining Wall on Spread Footings**
Concrete L-Shaped walls are cantilever walls without heels. These walls are used when there are space limitations for cantilever walls. Since there is no heel the height of these walls is limited to about 7 feet depending on the soil conditions and the slope of the retained soil.

L-Shaped Walls are often used next to roadways where the footings are frequently used as shoulders and where the wall will require structural capacity for collision forces.
1.2 Loads

Dead Loads

Dead loads shall be determined from the Weight of Materials Table of the Loads Section in the Bridge Manual.

Equivalent Fluid Pressure (Earth Pressures)

AASHTO 3.20.1

For determining equivalent earth pressures for Group Loadings I through VI the Rankine Formula for Active Earth Pressure shall be used.

Rankine Formula: \[ P_a = \frac{1}{2} C_a \gamma_s H^2 \]

where:

\[ C_a = \cos \left( \frac{\cos \delta - \sqrt{\cos^2 \delta - \cos^2 \phi}}{\cos \delta + \sqrt{\cos^2 \delta - \cos^2 \phi}} \right) \]

- \( P_a \) = equivalent active earth pressure
- \( C_a \) = coefficient of active earth pressure
- \( H \) = height of the soil face at the vertical plane of interest
- \( \gamma_s \) = unit weight of soil
- \( \delta \) = slope of fill in degrees
- \( \phi \) = angle of internal friction of soil in degrees

Example

Given

\( \delta = 3:1 \) (H:V) slope
\( \phi = 25^\circ \)
\( \gamma_s = 0.120 \) kcf
\( H = 10 \) ft

\( \delta = \arctan \left( \frac{1}{3} \right) = 18.4^\circ \)

\[ C_a = \cos(18.4^\circ) \left( \frac{\cos(18.4^\circ) - \sqrt{\cos^2(18.4^\circ) - \cos^2(25^\circ)}}{\cos(18.4^\circ) + \sqrt{\cos^2(18.4^\circ) - \cos^2(25^\circ)}} \right) = 0.515 \]

\[ P_a = (1/2)(0.515)(0.120 \text{ kips/ft}^3)(10 \text{ ft})^2 = 3.090 \text{ kips per foot of wall length} \]
The $\phi$ angle shall be determined by the Materials Division from soil tests. If the $\phi$ angle cannot be provided by the Materials Division a $\phi$ angle of 27 degrees shall be used.

Drainage shall be provided to relieve water pressure from behind all cast-in-place concrete retaining walls. If adequate drainage cannot be provided then walls shall be designed to resist the maximum anticipated water pressure.

**Surcharge Due to Point, Line, and Strip Loads**

Surcharge due to point and line loads on the soil being retained shall be included as dead load surcharge. The effect of these loads on the wall may be calculated using Figure 5.5.2B from AASHTO.

Surcharge due to strip loads on the soil being retained shall be included as a dead load surcharge load. The following procedure as described in *Principles of Foundation Engineering* by Braja M. Das (1995) shall be applied to calculate these loads when strip loads are applicable. An example of this application is when a retaining wall is used in front of an abutment so that the wall is retaining the soil from behind the abutment as a strip load on the soil being retained by the wall.

**RETAINING WALL IN FRONT OF AN ABUTMENT**

The portion of soil that is in the active wedge must be determined because the surcharge pressure only affects the wall if it acts on the active wedge. The actual failure surface in the backfill for the active state can be represented by ABC shown in the figure below. An approximation to the failure surface based on Rankine's active state is shown by dashed line AD. This approximation is slightly unconservative because it neglects friction at the pseudo-wall to soil interface.

The following variables are shown in the figure below:

- $\beta$ = slope of the active failure plane in degrees
- $\delta$ = slope of fill in degrees
- $H$ = height of fill (from the bottom of the footing).
- $L_1$ = distance from back of stem to back of footing heel
- $L_2$ = distance from footing heel to intersection of failure plane with ground surface
DETERMINATION OF ACTIVE WEDGES

In order to determine $\beta$, the following equation which has been derived from Rankine's active earth pressure theory must be solved by iteration:

$$\tan(-\beta) + \frac{1}{\tan(\beta - \phi)} - \frac{1}{\tan(\beta - \delta)} + \frac{1}{\tan(90^\circ + \phi + \delta - \beta)} = 0$$

$\phi$ = angle of internal friction of soil in degrees

A good estimate for the first iteration is to let $\beta = 45^\circ + (\phi/2)$. In lieu of iterating the above equation a conservative estimate for $\beta$ is 45 degrees. Once $\beta$ has been established, an estimate of $L_1$ is needed to determine $L_2$. From the geometry of the variables shown in the above figure:

$$L_2 = H \frac{\cos \delta \cos \beta}{\sin(\beta - \delta)}$$

The resultant pressure due to the strip load surcharge and its location are then determined. The following variables are shown in the figure below:

$q$ = load per unit area

$P_s$ = resultant pressure on wall due only to surcharge earth pressure

$z$ = location of $P_s$ measured from the bottom of the footing

$L_3$ = distance from back of stem to where surcharge pressure begins
SURCHARGE PRESSURE ON RETAINING WALL

From the figure:

\[ P_s = \frac{q}{90} \left[ H \left( \theta_2 - \theta_1 \right) \right] \]

where

\[ \theta_1 = \arctan \left( \frac{L_2}{H} \right) \] and \[ \theta_2 = \arctan \left( \frac{L_2}{H} \right) \]

\[ z = \frac{H^2 (\theta_2 - \theta_1) - (R - Q) + 57.30L_4H}{2H(\theta_2 - \theta_1)} \]

where

\[ R = (L_2)^2 (90^\circ - \theta_2) \] and \[ Q = (L_3)^2 (90^\circ - \theta_1) \]

When applicable, \( P_s \) is applied to the wall in addition to other earth pressures. The wall is then designed as usual.

**Live Load Surcharge**

**AASHTO 3.20.3 & 5.5.2**

Live load surcharge pressure of not less than two feet of earth shall be applied to the structure when highway traffic can come within a horizontal distance equal to one-half of the wall height, measured from the plane where earth pressure is applied.
**General**

**LIVE LOAD SURCHARGE**

\[ P_{LLS} = (2 \text{ ft}) \gamma_s C_a H \]

- \( P_{LLS} \) = pressure due to live load surcharge only
- \( \gamma_s \) = unit weight of soil (Note: AASHTO 5.5.2 specifies a minimum of 125 pcf for live load surcharge, MoDOT policy allows 120 pcf as given from the Weight of Materials Table of the Loads Section in the Bridge Manual.)
- \( C_a \) = coefficient of active earth pressure
- \( H \) = height of the soil face at the vertical plane of interest

The vertical live load surcharge pressure should only be considered when checking footing bearing pressures, when designing footing reinforcement, and when collision loads are present.

**Live Load Wheel Lines**

Live load wheel lines shall be applied to the footing when the footing is used as a riding or parking surface.

**AASHTO 3.24.5.1.1 & 5.5.6.1**

Distribute a \( LL_{WL} \) equal to 16 kips as a strip load on the footing in the following manner.

\[ P = \frac{LL_{WL}}{E} \]

where \( E = 0.8X + 3.75 \)

\( X \) = distance in feet from the load to the front face of the wall

**AASHTO 3.24.2 & 3.30**

The wheel lines shall move 1 foot from the barrier curb or wall to 1 foot from the toe of the footing.
Collision Forces

Collision forces shall be applied to a wall that can be hit by traffic. Apply a point load of 10 kips to the wall at a point 3 ft above the finished ground line.

SECTION

Distribute the force to the wall in the following manner:

\[
\text{Force per ft of wall} = \frac{10 \text{ kips}}{2L}
\]

PROFILE

When considering collision loads, a 25% overstress is allowed for bearing pressures and a factor of safety of 1.2 shall be used for sliding and overturning.

Wind and Temperature Forces

These forces shall be disregarded except for special cases, consult the Structural Project Manager.

When walls are greater than 84 feet long, an expansion joint shall be provided. Contraction joint spacing shall not exceed 28 feet.

Seismic Loads

Retaining walls in Seismic Performance Category A (SPC A) and SPC B which are located adjacent to roadways may be designed in accordance with AASHTO specifications for SPC A. Retaining walls in SPC B which are located under a bridge abutment or in a location where failure of the wall may affect the structural integrity of a bridge shall be designed to AASHTO specifications for SPC B. All
retaining walls located in SPC C and SPC D shall be designed in accordance to AASHTO specifications for the corresponding SPC.

In seismic category B, C and D determine equivalent fluid pressure from Mononobe-Okabe static method.

\[ P_{AE} = \text{equivalent active earth pressure during an earthquake} \]

\[ P_{AE} = \frac{1}{2} \gamma_s H^2 (1 - k_v) K_{AE} \]

where

\[ K_{AE} = \text{seismic active pressure coefficient} \]

\[ K_{AE} = \frac{\cos^2(\phi - \theta - \beta)}{\cos \theta \cos^2 \beta \cos(\delta + \beta + \theta)} \left( 1 + \frac{\sin(\phi + \delta)\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta)\cos(i - \beta)} \right)^2 \]

\[ \gamma_s = \text{unit weight of soil} \]

\[ k_v = \text{vertical acceleration coefficient} \]

\[ k_h = \text{horizontal acceleration coefficient which is equal to 0.5A for all walls, but 1.5A for walls with battered piles where} \]

\[ A = \text{seismic acceleration coefficient} \]

The following variables are shown in the figure below:

\[ \phi = \text{angle of internal friction of soil} \]

\[ \theta = \arctan \left( \frac{k_h}{1 - k_v} \right) \]

\[ \beta = \text{slope of soil face} \]

\[ \delta = \text{angle of friction between soil and wall in degrees} \]

\[ i = \text{backfill slope angle in degrees} \]

\[ H = \text{distance from the bottom of the part of the wall to which the pressure is applied to the top of the fill at the location where the earth pressure is to be found.} \]
Group Loads
For SPC A and B (if wall does not support an abutment), apply AASHTO Group I Loads only. Bearing capacity, stability and sliding shall be calculated using working stress loads. Reinforced concrete design shall be calculated using load factor design loads.

AASHTO Table 3.22.1A

AASHTO Group I Load Factors for Load Factor Design of concrete:
\[ \gamma = 1.3 \]
\[ \beta_D = 1.0 \text{ for concrete weight} \]
\[ \beta_D = 1.0 \text{ for flexural member} \]
\[ \beta_E = 1.3 \text{ for lateral earth pressure for retaining walls} \]
\[ \beta_E = 1.0 \text{ for vertical earth pressure} \]
\[ \beta_{LL} = 1.67 \text{ for live load wheel lines} \]
\[ \beta_{LL} = 1.67 \text{ for collision forces} \]

AASHTO 5.14.2

\[ \beta_E = 1.67 \text{ for vertical earth pressure resulting from live load surcharge} \]
\[ \beta_E = 1.3 \text{ for horizontal earth pressure resulting from live load surcharge} \]

For SPC B (if wall supports an abutment), C, and D apply AASHTO Group I Loads and seismic loads in accordance with AASHTO Division IA - Seismic Design Specifications.

AASHTO Div. IA 4.7.3

When seismic loads are considered, load factor for all loads = 1.0.
3.62.2 Mechanically Stabilized Earth (MSE) Walls

2.1 Design

Designs of Mechanically Stabilized Earth (MSE) walls are completed by consultants or contractors in accordance with Section 5 of the AASHTO Specifications. MoDOT Internet site contains a listing of facing unit manufacturers, soil reinforcement suppliers, and wall system suppliers which have been approved for use. See Sec 720 and 1010 of Missouri Standard Specifications for additional information. Geotechnical Section is responsible for checking global stability, which should be reported on the Foundation Investigation Geotechnical Report. For MSE wall preliminary information, see EPG 751.1.4.3 MSE Walls.

General policy

- Small block walls are limited to a 10 foot height in one lift.
- For small block walls, top cap units shall be used and shall be permanently attached by means of a resin anchor system.
- For large block walls, capstone may be substituted for coping and either shall be permanently attached to wall by panel dowels.
- MSE walls shall not be used where exposure to acid water may occur such as in areas of coal mining.
- MSE walls shall not be used where scour is a problem.
- MSE walls with metallic soil reinforcement shall not be used where stray electrical ground currents may occur as would be present near electrical substations.
- No utilities shall be allowed in the reinforced earth if future access to the utilities would require that the reinforcement layers be cut, or if there is a potential for material, which can cause degradation of the soil reinforcement, to leak out of the utilities into the wall backfill, with the exception of storm water drainage.
- The interior angle between two walls must be greater than 70 degrees.
- Small block walls may be battered up to 1.5 inches per foot.
- The friction angle used for the computation of horizontal forces within the reinforced soil shall be greater than or equal to 34 degrees.
- All reinforcement shall be epoxy coated in the concrete face for walls subject to spraying from adjacent roadways (approximately 10 feet or less from the curb.)
- All concrete except facing panels or units shall be CLASS B or B-1.
- The friction angle of the soil to be retained by the reinforced earth shall be listed on the plans as well as the friction angle for the foundation material the wall is to rest on.
- Seismic performance category and acceleration coefficient shall be listed on the plans.
- Factors of Safety for MSE walls shall be 2.0 for overturning, 1.5 for sliding, 2.0 for ultimate bearing capacity and 1.5 for pullout resistance.
- Factors of Safety for seismic design shall be 1.5 for overturning and 1.1 for sliding.
- Gutter type should be selected at the core team meeting.
- When gutter is required without fencing, use Type A or Type B gutter (for detail, see Mo. Std. Plan 609.00).
• When gutter is required with fencing, use Modified Type A or Modified Type B gutter (for detail, see Mo. Std. Plan 607.11).
• When fencing is required without gutter, place in tube and grout behind the MSE wall (for detail, see Page 2.2-5).
• Do not use small block walls in the following locations:

  Within the splash zone from snow removal operations (assumed to be 15 feet from the edge of the shoulder).

  Where the blocks will be continuously wetted, such as around sources of water.

  Where blocks will be located behind barrier curbs or other obstacles, which will trap salt-laden snow from removal operations.

  For structurally critical applications, such as containing necessary fill around structures.

  In tiered wall systems.

• For locations where small block walls are not desirable, consider coloring agents and/or architectural forms using large block walls for aesthetic installations.

• Drainage pipes for all large and small block walls shall be a minimum of a 6" diameter perforated PVC or PE pipe (See Sec 1013) unless larger sizes are required by design by the wall manufacturer. Show drainage pipe size on plans. Screens should be installed and maintained on drain pipe outlets. Outlet screens and cleanouts should be detailed (shown on construction drawing).

### MSE Wall Construction:

#### Corrugated Metal Pipe Pile Spacers Guidance:

Corrugated metal pipe pile spacers (CMPPS) shall be used at pile locations behind mechanically stabilized earth walls to protect the wall reinforcement when driving pile for the bridge substructure at end bents(s). CMPPS shall have an inside diameter greater than that of the pile and large enough to avoid damage to the pipe when driving the pile. The bottom of the CMPPS shall be placed 5’ min. below the bottom of the MSE wall leveling pad. The pipe shall be filled with sand or other approved material after the pile is placed and before driving. CMPPS shall be accurately located and capped for future pile construction.

Alternatively, the contractor shall be given the option of driving the piles before construction of the retaining wall and placing the wall reinforcing and backfill material around the piling. The contractor shall adequately support the piling to insure that proper pile alignment is maintained during the wall construction. The contractor’s plan for bracing the pile shall be submitted to the engineer for review. Piling shall be designed for downdrag (DD) loads due to either method. Oversized CMPPS with sand placed after driving may be considered to mitigate some of the effects of downdrag (DD) loads. Oversized CMPPS shall account...
for pile size, thermal movements of the bridge, pile placement plan, and vertical and horizontal placement tolerances.

The minimum clearance from the back face of MSE walls to the front face of the end bent beam shall be 3'-9" (Typ.). The 3'-9" dimension is based on the use of 18" CMPPS & FHWA-NHI-10-24, Figure 5-17C, which will help ensure that soil reinforcement is not skewed more than 15° for nut and bolt reinforcement connections. Other types of connections may require different methods for splaying. In the event that the 3'-9" dimension or setback cannot be used, the following guidance for CMPPS clearance shall be used: CMPPS shall be placed 18" clear min. from the back face of MSE wall panels; 12" minimum clearance is required between CMPPS and leveling pad and 18" minimum clearance is required between leveling pad and pile.

**MSE Wall Plan and Geometrics**

- A plan view shall be drawn showing a baseline or centerline, roadway stations and wall offsets. The plan shall contain enough information to properly locate the wall. The ultimate right of way shall also be shown, unless it is of a significant distance from the wall and will have no bearing on the wall design or construction.
- Stations and offsets are established between one construction baseline or roadway centerline and a wall control line (baseline). Some wall designs contain a slight batter, while others are vertical. A wall control line is set at the front face of the wall, either along the top or at the base of the wall, whichever is critical to the proposed improvements. For battered walls, to allow for batter adjustments of the stepped level pad or variation of the top of the wall, the wall control line (baseline) is to be shown at a fixed elevation. For battered walls, the offset location and elevation of control line shall be indicated. All horizontal breaks in the wall are given station-offset points, and walls with curvature indicate station-offsets to the PC and PT of the wall, and the radius.
- Any obstacles which may possibly interfere with wall reinforcing strips are shown. Drainage structures, lighting, or truss pedestals and footings, etc. are to be shown, with station offset to centerline of the obstacle, with obstacle size. Skew angles are shown to indicate the angle between a wall and a pipe or box which runs through the wall.
- Elevations at the top and bottom of the wall shall be shown at 25 foot intervals and at any break points in the wall.
- Curve data and/or offsets shall be shown at all changes in horizontal alignment. If battered wall systems are used on curved structures, show offsets at 10 foot (max.) intervals from the baseline.
- Details of any architectural finishes (formliners, concrete coloring, etc.).
- Details of threaded rod connecting the top cap block.
- Estimated quantities, total sq. ft. of mechanically stabilized earth systems.
• Proposed grade and theoretical top of leveling pad elevation shall be shown in constant slope. Slope line shall be adjusted per project. Top of wall or coping elevation and stationing shall be shown in the developed elevation per project. If leveling pad is anticipated to encounter rock, then contact the Geotechnical Section for leveling pad minimum embedment requirements.

MSE Wall Cross Sections

• A typical wall section for general information is shown.
• Additional sections are drawn for any special criteria. The front face of the wall is drawn vertical, regardless of the wall type.
• Any fencing and barrier curb are shown.
• Barriers if needed are shown on the cross section. Concrete barriers are attached to the roadway or shoulder pavement, not to the MSE wall. Standard Type B barrier curbs are placed along wall faces when traffic has access to the front face of the wall over shoulders of paved areas.
Note: Leveling Pad not shown for clarity.

The designer shall show on the plans the minimum embedment = max (2', embedment based on Geotechnical Report and global stability requirements). Minimum embedment shall be provided in accordance with AASHTO 5.8.1 & Geotechnical Report.

* Minimum 6" diameter perforated PVC or PE pipe, unless larger size pipes are required by design by wall manufacturer.

** Topmost layer of reinforcement shall be fully covered with select granular backfill for structural systems, as approved by the wall manufacturer, before placement of the Separation Geotextile.

*** Inverted U-shape reinforced capstone may be used in lieu of coping. Panel dowels for capstone shall be required and as provided by manufacturer.

TYPICAL SECTION THRU GENERIC LARGE BLOCK WALL

The designer shall show on the plans the minimum embedment = max (2', embedment based on Geotechnical Report and global stability requirements). Minimum embedment shall be provided in accordance with AASHTO 5.8.1 & Geotechnical Report.

* Minimum 6" diameter perforated PVC or PE pipe, unless larger size pipes are required by design by wall manufacturer.

** Topmost layer of reinforcement shall be fully covered with select granular backfill for structural systems, as approved by the wall manufacturer, before placement of the Separation Geotextile.

TYPICAL SECTION THRU GENERIC SMALL BLOCK WALL

CAPSTONE ANCHOR DETAILS

**Retaining Walls - Section 3.62**

**MSE Walls**

Reinforced coping shall be attached to wall by panel dowels (coping only at large block wall).

A minimum horizontal bench 4'-0" wide shall be provided in front of the wall.

The designer shall show on the plans the minimum embedment = max (2', embedment based on Geotechnical Report and global stability requirements). Minimum embedment shall be provided in accordance with AASHTO 5.8.1 & Geotechnical Report.

Inverted U-shape reinforced capstone may be used in lieu of coping. Panel dowels for capstone shall be required and as provided by manufacturer.

**TYPICAL SECTION THRU ANY MSE WALL FOUNDED ON SLOPE**

**WALL PROFILE**

Note:

Holes are 5/8" round, extend 4" into the third layer of blocks, recessed 2" deep by 1-1/2" round.

Rods or reinforcing bars are secured by an approved resin anchor system in accordance with Sec 1039.

Recess hole to be backfilled with non-shrink cement grout.

**WALL PLAN**

**Details of 1/2" Threaded Rod or Reinforcing Rod**

**Effective: Feb. 2010**

**Supersedes: May 2004**
BATTERED SMALL BLOCK WALLS

Battered mechanically stabilized earth wall systems may be used unless the design layout specifically calls for a vertical wall (large block walls shall not be battered and small block walls may be built vertical). If a battered MSE wall system is allowed, then the following note shall be placed on the design plans:

"The top and bottom of wall elevations are given for a vertical wall. If a battered small block wall system is used, the height of the wall shall be adjusted as necessary to fit the ground slope. If fence is built on an extended gutter, then the height of the wall shall be adjusted further."

For battered walls, note on the plans whether the horizontal offset from the baseline is fixed at the top or bottom of the wall. Horizontal offset and corresponding vertical elevation shall be noted on plans.
FENCING

Notes:

Fencing may be installed on the Modified Type A or Modified Type B Gutter or behind the MSE Wall.

For Modified Type A and Modified Type B Gutter and Fence Post Connection details, see Missouri Standard Plans No. 607.11.

For Fence Post Connection Behind MSE Wall, see detail below.
3.62.3 Cast-In-Place Concrete Retaining Walls

3.1 Unit Stresses

**Concrete**
Concrete for retaining walls shall be Class B Concrete (f'c = 3000 psi) unless the footing is used as a riding surface in which case Class B-1 Concrete (f'c = 4000 psi) shall be used.

**Reinforcing Steel**
Reinforcing Steel shall be Grade 60 (f_y = 60,000 psi).

**Pile Footing**
For piling capacities, see the Unit Stresses and Piling Sections of the Bridge Manual.

**Spread Footing**
For foundation material capacity, see the Unit Stresses Section of the Bridge Manual and the Design Layout Sheet.
3.2 Design

If the height of the wall or fill is a variable dimension, then base the structural design of the wall, toe, and heel on the high quarter point between expansion joints.

![Diagram of Distance between Expansion Joints and High Quarter Point]

Spread Footings

Location of Resultant

**AASHTO 5.5.5**

The resultant of the footing pressure must be within the section of the footing specified in the following table.

<table>
<thead>
<tr>
<th>When Retaining Wall is Built on:</th>
<th>AASHTO Group Loads I-VI</th>
<th>For Seismic Loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil ( a )</td>
<td>Middle 1/3</td>
<td>Middle 1/2 ( b )</td>
</tr>
<tr>
<td>Rock ( c )</td>
<td>Middle 1/2</td>
<td>Middle 2/3</td>
</tr>
</tbody>
</table>

a. Soil is defined as clay, clay and boulders, cemented gravel, soft shale, etc. with allowable bearing values less than 6 tons/sq. ft.

b. MoDOT is more conservative than AASHTO in this requirement.

c. Rock is defined as rock or hard shale with allowable bearing values of 6 tons/sq. ft. or more.

Note:
The location of the resultant is not critical when considering collision loads.
**Factor of Safety Against Overturning**

AASHTO 5.5.5

AASHTO Group Loads I - VI:
- F.S. for overturning ≥ 2.0 for footings on soil.
- F.S. for overturning ≥ 1.5 for footings on rock.

For seismic loading, F.S. for overturning may be reduced to 75% of the value for AASHTO Group Loads I - VI. For seismic loading:
- F.S. for overturning ≥ (0.75)(2.0) = 1.5 for footings on soil.
- F.S. for overturning ≥ (0.75)(1.5) = 1.125 for footings on rock.

For collision forces:
- F.S. for overturning ≥ 1.2.

**Factor of Safety Against Sliding**

AASHTO 5.5.5

Only spread footings on soil need be checked for sliding because spread footings on rock or shale are embedded into the rock.
- F.S. for sliding ≥ 1.5 for AASHTO Group Loads I - VI.
- F.S. for sliding ≥ (0.75)(1.5) = 1.125 for seismic loads.
- F.S. for sliding ≥ 1.2 for collision forces.

The resistance to sliding may be increased by:
- adding a shear key that projects into the soil below the footing.
- widening the footing to increase the weight and therefore increase the frictional resistance to sliding.

**Passive Resistance of Soil to Lateral Load**

The Rankine formula for passive pressure can be used to determine the passive resistance of soil to the lateral force on the wall. This passive pressure is developed at shear keys in retaining walls and at end abutments.

The passive pressure against the front face of the wall and the footing of a retaining wall is loosely compacted and should be neglected when considering sliding.

Rankine formula: \( P_p = \frac{1}{2} C_p \gamma_S \left[ H^2 - H_1^2 \right] \) where the following variables are defined in the figure below:

\[
C_p = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right)
\]
\[
\gamma_1 = \frac{2}{3} \left( \frac{y_2^2}{H^2 - H_1^2} \right)
\]
\[
P_p = \text{passive force at shear key in pounds per foot of wall length}
\]
\[
C_p = \text{coefficient of passive earth pressure}
\]
\[
\gamma_S = \text{unit weight of soil}
\]
\[
H = \text{height of front face fill less 1 foot min. for erosion}
\]
\[
H_1 = H - \text{depth of shear key}
\]
\[
y_1 = \text{location of } P_p \text{ from bottom of footing}
\]
\[
\phi = \text{angle of internal friction of soil}
\]
The resistance due to passive pressure in front of the shear key shall be neglected unless the key extends below the depth of frost penetration.

MoDOT Materials Division

Frost line is set at 36 inches at the north border of Missouri and at 18” at the south border.

Passive Pressure During Seismic Loading

During an earthquake, the passive resistance of soil to lateral loads is slightly decreased. The Mononobe-Okabe static method is used to determine the equivalent fluid pressure.

\[ P_{PE} = \text{equivalent passive earth pressure during an earthquake} \]

1992 AASHTO Div.1A Eqn. C6-5

\[ P_{PE} = \frac{1}{2} \gamma_S H^2 \left(1 - k_V\right) K_{PE} \]

where:

1992 AASHTO Div.1A Eqn. C6-6

\[ K_{PE} = \text{seismic passive pressure coefficient} \]

\[ K_{PE} = \frac{\cos^2(\phi - \theta + \beta)}{\cos \theta \cos^2 \beta \cos(\delta - \beta + \theta) \left[1 - \frac{\sin(\phi + \delta) \sin(\phi - \theta + i)}{\cos(\delta - \beta + \theta) \cos(i - \beta)}\right]^2} \]

\[ \gamma_S = \text{unit weight of soil} \]

\[ H = \text{height of soil at the location where the earth pressure is to be found} \]

\[ k_V = \text{vertical acceleration coefficient} \]

\[ \phi = \text{angle of internal friction of soil} \]

\[ \theta = \arctan \left[ \frac{k_h}{1 - k_V} \right] \]

\[ k_H = \text{horizontal acceleration coefficient} \]

\[ \beta = \text{slope of soil face in degrees} \]
i = backfill slope angle in degrees
\( \delta \) = angle of friction between soil and wall

**Special Soil Conditions**

Due to creep, some soft clay soils have no passive resistance under a continuing load. Removal of undesirable material and replacement with suitable material such as sand or crushed stone is necessary in such cases. Generally, this condition is indicated by a void ratio above 0.9, an angle of internal friction (\( \phi \)) less than 22°, or a soil shear less than 0.8 ksf. Soil shear is determined from a standard penetration test.

\[
\text{Soil Shear} = \frac{k}{i^2} = \frac{\text{blows per 12 in}}{10}
\]

**Friction**

In the absence of tests, the total shearing resistance to lateral loads between the footing and a soil that derives most of its strength from internal friction may be taken as the normal force times a coefficient of friction. If the plane at which frictional resistance is evaluated is not below the frost line then this resistance must be neglected.

**When A Shear Key Is Not Used**

Sliding is resisted by the friction force developed at the interface between the soil and the concrete footing along the failure plane. The coefficient of friction for soil against concrete can be taken from the table below. If soil data is not readily available or is inconsistent, the friction factor (f) can be taken as

\[
f = \tan \left( \frac{2}{3} \phi \right)
\]


**AASHTO Table 5.5.2B**

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Coefficient of Friction</th>
</tr>
</thead>
<tbody>
<tr>
<td>coarse-grained soil without silt</td>
<td>0.55</td>
</tr>
<tr>
<td>coarse-grained soil with silt</td>
<td>0.45</td>
</tr>
<tr>
<td>Silt (only)</td>
<td>0.35</td>
</tr>
<tr>
<td>clay</td>
<td>0.30 (^h)</td>
</tr>
</tbody>
</table>
a. It is not necessary to check rock or shale for sliding due to embedment.
b. Caution should be used with soils with $\phi < 22^\circ$ or soil shear < 0.8 k/sq.ft. (soft clay soils). Removal and replacement of such soil with suitable material should be considered.

When A Shear Key Is Used

When a shear key is used, the failure plane is located at the bottom of the shear key in the front half of the footing. The friction force resisting sliding in front of the shear key is provided at the interface between the stationary layer of soil and the moving layer of soil, thus the friction angle is the internal angle of friction of the soil (soil against soil). The friction force resisting sliding on the rest of the footing is of that between the concrete and soil. Theoretically the bearing pressure distribution should be used to determine how much normal load exists on each surface, however it is reasonable to assume a constant distribution. Thus the normal load to each surface can be divided out between the two surfaces based on the fractional length of each and the total frictional force will be the sum of the normal load on each surface multiplied by the corresponding friction factor.

Bearing Pressure

Group Loads I - VI

*AASHTO 4.4.7.1.2 & 4.4.8.1.3*

The bearing capacity failure factor of safety for Group Loads I - VI must be greater than or equal to 3.0. This factor of safety is figured into the allowable bearing pressure given on the "Design Layout Sheet".

The bearing pressure on the supporting soil shall not be greater than the allowable bearing pressure given on the "Design Layout Sheet".

Seismic Loads

*AASHTO Div. IA 6.3.1(B)*

When seismic loads are considered, AASHTO allows the ultimate bearing capacity to be used. The ultimate capacity of the foundation soil can be conservatively estimated as 2.0 times the allowable bearing pressure given on the "Design Layout".
Stem Design

AASHTO 5.5.6.2

The vertical stem (the wall portion) of a cantilever retaining wall shall be designed as a cantilever supported at the base.

Footing Design

AASHTO 5.5.6.1

Toe

The toe of the base slab of a cantilever wall shall be designed as a cantilever supported by the wall. The critical section for bending moments shall be taken at the front face of the stem. The critical section for shear shall be taken at a distance \( d \) (\( d \) = effective depth) from the front face of the stem.

Heel

The rear projection (heel) of the base slab shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used. The heel shall be designed as a cantilever supported by the wall. The critical section for bending moments and shear shall be taken at the back face of the stem.

Shear Key Design

The shear key shall be designed as a cantilever supported at the bottom of the footing.

Pile Footings

Footings shall be cast on piles when specified on the "Design Layout Sheet". If the horizontal force against the retaining wall cannot otherwise be resisted, some of the piles shall be driven on a batter.

Pile Arrangement

For retaining walls subject to moderate horizontal loads (walls 15 to 20 feet high), the following layout is suggested.
For higher walls and more extreme conditions of loading, it may be necessary to:

- use the same number of piles along all rows
- use three rows of piles
- provide batter piles in more than one row

**Loading Combinations for Stability and Bearing**

The following table gives the loading combinations to be checked for stability and pile loads. These abbreviations are used in the table:

- DL = dead load weight of the wall elements
- SUR = two feet of live load surcharge
- E = earth weight
- EP = equivalent fluid earth pressure
- COL = collision force
- EQ = earthquake inertial force of failure wedge

<table>
<thead>
<tr>
<th>Loading Case</th>
<th>Vertical Loads</th>
<th>Horizontal Loads</th>
<th>Overturning Factor of Safety</th>
<th>Battered Toe Piles</th>
<th>Vertical Toe Piles</th>
</tr>
</thead>
<tbody>
<tr>
<td>I&lt;sup&gt;a&lt;/sup&gt;</td>
<td>DL+SUR+E</td>
<td>EP+SUR</td>
<td>1.5</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>II</td>
<td>DL+SUR+E</td>
<td>EP+SUR+COL</td>
<td>1.2</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>III</td>
<td>DL+E</td>
<td>EP</td>
<td>1.5</td>
<td>1.5</td>
<td>2.0</td>
</tr>
<tr>
<td>IV&lt;sup&gt;b&lt;/sup&gt;</td>
<td>DL+E</td>
<td>None</td>
<td>-----</td>
<td>-----</td>
<td>-----</td>
</tr>
<tr>
<td>V&lt;sup&gt;c&lt;/sup&gt;</td>
<td>DL+E</td>
<td>EP+EQ</td>
<td>1.125</td>
<td>1.125</td>
<td>1.5</td>
</tr>
</tbody>
</table>

a. Load Case I should be checked with and without the vertical surcharge.
b. A 25% overstress is allowed on the heel pile in Load Case IV.
c. The factors of safety for earthquake loading are 75% of that used in Load Case III. Battered piles are not recommended for use in seismic performance categories B, C, and D. Seismic design of retaining walls is not required in SPC A and B. Retaining walls in SPC B located under a bridge abutment shall be designed to AASHTO Specifications for SPC B.
Pile Properties and Capacities

For Load Cases I-IV in the table above, the allowable compressive pile force may be taken from the pile capacity table in the Piling Section of the Bridge Manual which is based in part on AASHTO 4.5.7.3. Alternatively, the allowable compressive pile capacity of a friction pile may be determined from the ultimate frictional and bearing capacity between the soil and pile divided by a safety factor of 3.5 (AASHTO Table 4.5.6.2.A). The maximum amount of tension allowed on a heel pile is 3 tons.

For Load Case V in the table above, the allowable compressive pile force may be taken from the pile capacity table in the Piling Section of the Bridge Manual multiplied by the appropriate factor (2.0 for steel bearing piles, 1.5 for friction piles). Alternatively, the allowable compressive pile capacity of a friction pile may be determined from the ultimate frictional and bearing capacity between the soil and pile divided by a safety factor of 2.0. The allowable tension force on a bearing or friction pile will be equal to the ultimate friction capacity between the soil and pile divided by a safety factor of 2.0.

To calculate the ultimate compressive or tensile capacity between the soil and pile requires the boring data which includes the SPT blow counts, the friction angle, the water level, and the soil layer descriptions.

Assume the vertical load carried by battered piles is the same as it would be if the pile were vertical. The properties of piles may be found in the Piling Section of the Bridge Manual.

Neutral Axis of Pile Group

Locate the neutral axis of the pile group in the repetitive strip from the toe of the footing at the bottom of the footing.

Moment of Inertia of Pile Group

The moment of inertia of the pile group in the repetitive strip about the neutral axis of the section may be determined using the parallel axis theorem:

\[ I = \sum (I_A) + \sum (Ad^2) \]

where:

- \( I_A \) = moment of inertia of a pile about its neutral axis
- \( A \) = area of a pile
- \( d \) = distance from a pile's neutral axis to pile group's neutral axis

\( I_A \) may be neglected so the equation reduces to:

\[ I = \sum (Ad^2) \]

Resistance To Sliding

Any frictional resistance to sliding shall be ignored, such as would occur between the bottom of the footing and the soil on a spread footing.
Friction or Bearing Piles With Batter (Case 1)
Retaining walls using friction or bearing piles with batter should develop lateral strength (resistance to sliding) first from the batter component of the pile and second from the passive pressure against the shear key and the piles.

Friction or Bearing Piles Without Batter (Case 2)
Retaining walls using friction or bearing piles without batter due to site constrictions should develop lateral strength first from the passive pressure against the shear key and second from the passive pressure against the pile below the bottom of footing. In this case, the shear key shall be placed at the front face of the footing.

Concrete Pedestal Piles or Drilled Shafts (Case 3)
Retaining walls using concrete pedestal piles should develop lateral strength first from passive pressure against the shear key and second from passive pressure against the pile below the bottom of the footing. In this case, the shear key shall be placed at the front of the footing. Do not batter concrete pedestal piles.

Resistance Due to Passive Pressure Against Pile
The procedure below may be used to determine the passive pressure resistance developed in the soil against the piles. The procedure assumes that the piles develop a local failure plane.

\[ F = \frac{1}{2} \gamma_s C_p H^2 B \]
\[ F = \frac{1}{2} \gamma_s C_p H^2 B \]
\[ \gamma_s \] = unit weight of soil
\[ H \] = depth of pile considered for lateral resistance (\( H_{\text{max}} = 6B \))
\[ C_p \] = coefficient of active earth pressure
\[ B \] = width of pile
\[ \phi \] = angle of internal friction of soil
Resistance Due to Pile Batter
Use the horizontal component (due to pile batter) of the allowable pile load as the lateral resistance of the battered pile. (This presupposes that sufficient lateral movement of the wall can take place before failure to develop the ultimate strength of both elements.)

\[ b = \text{the amount of batter per 12 inches.} \]
\[ c = \sqrt{(12 \text{ in})^2 + (b)^2} \]
\[ P_{HBatter} = P_T \left( \frac{b}{c} \right) (\# \text{ of battered piles}) \text{ where:} \]
\[ P_{HBatter} = \text{the horizontal force due to the battered piles} \]
\[ P_T = \text{the allowable pile load} \]
Maximum batter is 4" per 12".

Resistance Due to Shear Keys
A shear key may be needed if the passive pressure against the piles and the horizontal force due to batter is not sufficient to attain the factor of safety against sliding. The passive pressure against the shear key on a pile footing is found in the same manner as for spread footings.
**Resistance to Overturning**

The resisting and overturning moments shall be computed at the centerline of the toe pile at a distance of 6B (where B is the width of the pile) below the bottom of the footing. A maximum of 3 tons of tension on each heel pile may be assumed to resist overturning. Any effects of passive pressure, either on the shear key or on the piles, which resist overturning, shall be ignored.

![Diagram showing overturning moments and pile loads](image)

**Pile Properties**

**Location of Resultant**

The location of the resultant shall be evaluated at the bottom of the footing and can be determined by the equation below:

\[ e = \frac{\sum M}{\sum V} \]

where:

- \( e \) = the distance between the resultant and the neutral axis of the pile group
- \( \sum M \) = the sum of the moments taken about the neutral axis of the pile group at the bottom of the footing
- \( \sum V \) = the sum of the vertical loads used in calculating the moment

**Pile Loads**

The loads on the pile can be determined as follows:

\[ P = \frac{\sum V}{A} \pm \frac{Mc}{I} \]

Where:

- \( P \) = the force on the pile
- \( A \) = the areas of all the piles being considered
- \( M \) = the moment of the resultant about the neutral axis
- \( c \) = distance from the neutral axis to the centerline of the pile being investigated
- \( I \) = the moment of inertia of the pile group
Stem Design

AASHTO 5.5.6.2
The vertical stem (the wall portion) of a cantilever retaining wall shall be designed as a cantilever supported at the base.

Footing Design

Toe

AASHTO 5.5.6.1
The toe of the base slab of a cantilever wall shall be designed as a cantilever supported by the wall. The critical section for bending moments shall be taken at the front face of the stem. The critical section for shear shall be taken at a distance $d$ ($d = \text{effective depth}$) from the front face of the stem.

Heel

AASHTO 5.5.6.1
The top reinforcement in the rear projection (heel) of the base slab shall be designed to support the entire weight of the superimposed materials plus any tension load in the heel piles (neglect compression loads in the pile), unless a more exact method is used. The bottom reinforcement in the heel of the base slab shall be designed to support the maximum compression load in the pile neglecting the weight of the superimposed materials. The heel shall be designed as a cantilever supported by the wall. The critical sections for bending moments and shear shall be taken at the back face of the stem.

Shear Key Design

The shear key shall be designed as a cantilever supported at the bottom of the footing.
Counterfort Walls

Assumptions:

1. Stability

The external stability of a counterfort retaining wall shall be determined in the same manner as described for cantilever retaining walls. Therefore refer to previous pages for the criteria for location of resultant, factor of safety for sliding, and bearing pressures.

2. Stem

\[ P = C_o \delta \]  
where: \[ C_o = \text{coefficient of active earth pressure} \]

Design the wall to support horizontal load from the earth pressure and the live load surcharge (if applicable) as outlined on the previous pages and as designated in AASHTO Article 3.22, except that maximum horizontal loads shall be the calculated equivalent fluid pressure at 3/4 height of wall \((0.75H)\) which shall be considered applied uniformly from the lower quarter point to the bottom of wall.

In addition, vertical steel in the fill face of the bottom quarter of the wall shall be that required by the vertical cantilever wall with the equivalent fluid pressure of that \((0.25H)\) height.

Maximum concrete stress shall be assumed as the greater of the two thus obtained.
(2) Stem (Cont.)

The application of these horizontal pressures shall be as follows:

COUNTERFORT WALL SECTION

Moments are to be determined by analysis as a continuous beam. The counterforts are to be spaced so as to cause positive and negative moments to be approximately equal.
(3) Counterfort

Counterforts shall be designed as T-beams, of which the web is the flange and the counterfort is the stem. For this reason the concrete stresses are normally low and will not control.

For the design of reinforcing steel in the back of the counterfort, the effective depth shall be the perpendicular distance from the front face of the wall (at point that moment is considered), to center of reinforcing steel. (See sketch)

(4) Footing

The footing of the counterfort walls shall be designed as a continuous beam of spans equal to the distance between the counterforts.

The rear projection of heel shall be designed to support the entire weight of the superimposed materials, unless a more exact method is used. Refer to AASHO-Article 5.5.6.

Divide footing (Transversely) into four (4) equal sections for design footing pressures.

Counterfort walls on pile are very rare and are to be treated as special cases. See Structural Project Manager.

(5) Sign-Board type walls

The Sign-Board type of retaining walls are a special case of the counterfort retaining walls. This type of wall is used where the soil conditions are such that the footings must be placed a great distance below the finished ground line. For this situation, the wall is discontinued approximately 12" below the finished ground line or below the frost line.

Due to the large depth of the counterforts, it may be more economical to use a smaller number of counterforts than would otherwise be used.
(5) Sign-Board type walls: (Continued)

All design assumptions that apply to counterfort walls will apply to
sign-board walls with the exception of the application of horizontal
forces for the stem (or wall design), and the footing design which shall be
as follows:

[Diagram of sign-board wall with labels and annotations]

Load taken by horizontal steel

(6) Footing

The individual footings shall be designed transversely as cantilevers
supported by the wall. Refer to AASHTO - Article Section 5.
3.3 Example 1: Spread Footing Cantilever Wall

**Assumptions**
- Retaining wall is under an abutment or in a location where failure of the wall may affect the structural integrity of a bridge. Therefore, it must be designed for SPC B.
- Design is for a unit length (1 foot) of wall.
- Sum moments about the toe at the bottom of the footing for overturning.
- For Group Loads I-VI loading:
  - F.S. for overturning $\geq 2.0$ for footings on soil.
  - F.S. for sliding $\geq 1.5$.
  - Resultant to be within middle 1/3 of footing.
- For earthquake loading:
  - F.S. for overturning $\geq 0.75(2.0) = 1.5$.
  - F.S. for sliding $\geq 0.75(1.5) = 1.125$.
  - Resultant to be within middle 1/2 of footing.
- Base of footing is below the frost line.
- Neglect top one foot of fill over toe when determining passive pressure and soil weight.
Use of a shear key shifts the failure plane to "B" where resistance to sliding is provided by passive pressure against the shear key, friction of soil along failure plane "B" in front of the key, and friction between soil and concrete along the footing behind the key.

- Soil cohesion along failure plane is neglected.
- Footings are designed as cantilevers supported by the wall.
  - Critical sections for bending are at the front and back faces of the wall.
  - Critical sections for shear are at the back face of the wall for the heel and at a distance d (effective depth) from the front face for the toe.
- Neglect soil weight above toe of footing in design of the toe.
- The wall is designed as a cantilever supported by the footing.
- Load factors for AASHTO Groups I - VI for design of concrete:
  - $\gamma = 1.3$.
  - $\beta_E = 1.3$ for horizontal earth pressure on retaining walls.
  - $\beta_E = 1.0$ for vertical earth pressure.
- Load factor for earthquake loads $= 1.0$.

### Lateral Pressures Without Earthquake

$$C_d = \cos \delta \left[ \frac{\cos \delta - \cos^2 \delta - \cos^2 \phi}{\cos \delta + \sqrt{\cos^2 \delta - \cos^2 \phi}} \right]$$

$$C_d = \cos 18.435^\circ \left[ \frac{\cos 18.435^\circ - \cos^2 18.435^\circ - \cos^2 24^\circ}{\cos 18.435^\circ + \sqrt{\cos^2 18.435^\circ - \cos^2 24^\circ}} \right] = 0.546$$

$$C_p = \tan^2 \left( 45^\circ + \frac{\phi}{2} \right) = \tan^2 \left( 45^\circ + \frac{24^\circ}{2} \right) = 2.371$$

$$P_A = \frac{1}{2} \left[ 0.120 \frac{k}{ft} \right] (1\text{ft})(0.546)(10.667\text{ft})^2 = 3.726k$$

$$P_P = \frac{1}{2} \left[ 0.120 \frac{k}{ft} \right] (1\text{ft})(2.371)(5.0)^2 - (2.5)^2 = 2.668k$$

$$P_{AV} = P_A \sin \delta = 3.726k \sin 18.435^\circ = 1.178k$$

$$P_{AH} = P_A \cos \delta = 3.726k \cos 18.435^\circ = 3.534k$$
Load | Area (ft²) | Force (k) | Arm (ft) | Moment (ft-k) \\
--- | --- | --- | --- | --- \\
(1) | (0.5)(6.667ft)(2.222ft) = 7.407 | 0.889 | 7.278 | 6.469 \\
(2) | (6.667ft)(6.944ft) = 46.296 | 5.556 | 6.167 | 34.259 \\
(3) | (0.833ft)(8.000ft) + (0.5)(0.083ft)(8.000ft) = 7.000 | 1.050 | 2.396 | 2.515 \\
(5) | (2.500ft)(1.000ft) = 2.500 | 0.375 | 2.500 | 0.938 \\
(6) | (1.000ft)(1.917ft) + (0.5)(0.010ft)(1.000ft) = 1.922 | 0.231 | 0.961 | 0.222 \\
Σ | ΣV = 10.239 | ΣMr = 54.556 \\
P_{AV} | 1.178 | 9.500 | 11.192 \\
Σ resisting | ΣV = 11.417 | ΣMr = 65.748 \\
P_{AH} | 3.534 | 3.556 | 12.567 \\
P_p | 2.668 | 1.389^a \\

a. The passive capacity at the shear key is ignored in overturning checks, since this capacity is considered in the factor of safety against sliding. It is assumed that a sliding and overturning failure will not occur simultaneously. The passive capacity at the shear key is developed only if the wall does slide.

\[ y = \frac{H_1y^2 + \frac{1}{2}y^3}{H^2 - H_1} = \frac{(2.5ft)(2.5ft)^2 + \frac{1}{2}(2.5ft)^3}{(5.0ft)^2 - (2.5ft)^2} = 1.389ft \]

**Overturning**

\[ F.S. = \frac{M_R}{M_{OT}} = \frac{65.748(ft-k)}{12.567(ft-k)} = 5.232 \geq 2.0 \text{ o.k.} \]

where: \( M_{OT} \) = overturning moment; \( M_R \) = resisting moment

**Resultant Eccentricity**

\[ \chi = \frac{(65.748 - 12.567)(ft-k)}{11.417k} = 4.658ft \]
\[ e = \frac{9.500ft}{2} - 4.658ft = 0.092ft \]
\[ \frac{L}{6} = \frac{9.500ft}{6} = 1.583ft > e \text{ o.k.} \]

**Sliding**

Check if shear key is required for Group Loads I-VI:

\[ F.S. = \frac{\Sigma V(tan\phi_{s-e})}{P_{AH}} = \frac{11.042k\tan\frac{2}{3}(24^\circ)}{3.534k} = 0.896 \text{ no good - shear key req'd} \]

where: \( \phi_{s-e} \) = angle of friction between soil and concrete = \( (2/3)\phi_{c-s} \)

\[ F.S. = \frac{P_p + \Sigma V(\frac{1}{2})\tan\phi_{s-e} + (\frac{1}{2})\tan\phi_{s-c}}{P_{AH}} \]
where: $\phi_{ss}$ = angle of internal friction of soil

$$F.S. = \frac{2.668k(11.417k)}{3.534k} \left( \frac{2n}{9.50k} \tan 24^\circ + \frac{7.50 n}{9.50k} \tan \left( \frac{2}{9}(24^\circ) \right) \right) = 1.789 \geq 1.5 \quad \text{o.k.}$$

**Footing Pressure**

$$P = \sum V \left[ 1 \pm 6e \right]$$

$P_H = $ pressure at heel 

$$P_H = \frac{11.417k}{(1ft/9.50ft)} \left[ 1 - \frac{6(0.092ft)}{9.50ft} \right] = 1.132 \frac{k}{ft^2}$$

$P_T = $ pressure at toe 

$$P_T = \frac{11.417k}{(1ft/9.50ft)} \left[ 1 + \frac{6(0.092ft)}{9.50ft} \right] = 1.272 \frac{k}{ft^2}$$

Allowable pressure $= 2 \frac{\text{tons}}{ft^2} = 4 \frac{k}{ft^2} \geq 1.272 \frac{k}{ft^2} \quad \text{o.k.}$

**Lateral Pressures With Earthquake**

$k_n = 0.5A = 0.5(0.1) = 0.05$

$k_v = 0$

$$\theta = \arctan \left[ \frac{k_n}{1-k_v} \right] = \arctan \left[ \frac{0.05}{1-0} \right] = 2.862^\circ$$

**Active Pressure on Psuedo-Wall**

$\delta = \phi = 24^\circ$ ($\delta$ is the angle of friction between the soil and the wall. In this case, $\delta = \phi$ because the soil wedge considered is next to the soil above the footing.)

$$i = 18.435^\circ$$

$$\beta = 0^\circ$$

$$K_{AE} = \frac{\cos^2(\phi-\theta-\beta)}{\cos \theta \cos^2 \beta \cos (\delta+\theta+\theta) \left[ 1+ \frac{\sin(\phi+\theta+\phi+\theta+\theta)}{\cos(\phi+\theta+\phi+\theta)} \right]^2}$$

$$K_{AE} = \frac{\cos^2(24^\circ-24^\circ-0^\circ)}{\cos(2.862^\circ)\cos^2(0^\circ)\cos(24^\circ+0^\circ+2.862^\circ) \left[ 1+ \frac{\sin(24^\circ+24^\circ)\sin(24^\circ+24^\circ+2.862^\circ-18.435^\circ)}{\cos(24^\circ+24^\circ+2.862^\circ)\cos(18.435^\circ-0^\circ)} \right]^2}$$

$$K_{AE} = 0.674$$

$$P_{AE} = \frac{1}{2} G_s H^2 (1-k_v) K_{AE}$$

$$P_{AE} = \frac{1}{2} \left[ 0.120 \frac{ft}{in} \right] (10.667ft)^2 (1ft)(1 - 0)(0.674) = 4.602k$$

$$P_{AEV} = P_{AE}(\sin \delta) = 4.602k(\sin 24^\circ) = 1.872k$$
\[ P_{AEH} = P_{AE} \cos \delta = 4.602k(\cos 24^\circ) = 4.204k \]
\[ P'_{AH} = P_{AEH} - P_{AH} = 4.204k - 3.534k = 0.670k \]
\[ P'_{AV} = P_{AEV} - P_{AV} = 1.872k - 1.178k = 0.694k \]

where: \( P'_{AH} \) and \( P'_{AV} \) are the seismic components of the active force.

### Passive Pressure on Shear Key

\[ \delta = \phi = 24^\circ \] (\( \delta = \phi \) because the soil wedge considered is assumed to form in front of the footing.)
\[ i = 0^\circ \]
\[ \beta = 0^\circ \]

\[
K_{PE} = \frac{\cos^2(\phi - \theta + \beta) \cos \theta \cos^2(\delta - \beta + \theta) \left( 1 - \frac{\sin(\phi - \delta) \sin(\phi - \theta + i)}{\cos(\delta - \beta + \theta) \cos(\beta - \theta)} \right)^2}{\cos(2.862^\circ) \cos^2(0^\circ) \cos(24^\circ - 0^\circ) \cos^2(2.862^\circ)}
\]

\[ K_{PE} = 0.976 \]

\[ P_{PE} = \frac{1}{2} \gamma s H^2 (1 - k_v) K_{PE} \]

\[ P_{PE} = \frac{1}{2} \times 0.120 \times 5^2 \left[ (5.0ft)^2 - (2.5ft)^2 \right] (1ft)(1 - 0)(0.976) = 1.098k \]

### Load Force (k) Arm (ft) Moment (ft-k)

<table>
<thead>
<tr>
<th>Load</th>
<th>Force (k)</th>
<th>Arm (ft)</th>
<th>Moment (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Sigma ) (1) thru (6)</td>
<td>10.239</td>
<td>9.500</td>
<td>54.556</td>
</tr>
<tr>
<td>( P_{AV} )</td>
<td>1.178</td>
<td>9.500</td>
<td>11.192</td>
</tr>
<tr>
<td>( P'_{AV} )</td>
<td>0.694</td>
<td>9.500</td>
<td>6.593</td>
</tr>
<tr>
<td>( \Sigma ) resisting</td>
<td>( \Sigma V = 12.111 )</td>
<td>( \Sigma M_{OT} = 72.341 )</td>
<td></td>
</tr>
<tr>
<td>( P_{AH} )</td>
<td>3.534</td>
<td>3.556</td>
<td>12.567</td>
</tr>
<tr>
<td>( P'_{AH} )</td>
<td>0.670</td>
<td>6.400(^a)</td>
<td>4.288</td>
</tr>
<tr>
<td>( P_{PEV} )</td>
<td>0.447(^b)</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>( P_{PEH} )</td>
<td>1.003(^b)</td>
<td>1.389(^c)</td>
<td>0.000</td>
</tr>
</tbody>
</table>

\( \Sigma M_{OT} = 16.855 \)

\( P_{AH} \) acts at 0.6H of the wedge face (1992 AASHTO Div. IA Commentary).

\( P_{PEV} \) and \( P_{PEH} \) are the components of \( P_{PE} \) with respect to \( \delta \) (the friction angle). \( P_{PE} \) does not contribute to overturning.

The line of action of \( P_{PEH} \) can be located as was done for \( P_{P} \).

### Overturning

\[ F.S. = \frac{72.341/(ft-k)}{16.855/(ft-k)} = 4.292 > 1.5 \quad \text{O.K.} \]
**Resultant Eccentricity**

\[ X = \frac{72.341 \text{ft} - 16.855 \text{ft}}{12.111 \text{ft}} = 4.581 \text{ft} \]

\[ e = \frac{9.5 \text{ft}}{12} - 4.581 \text{ft} = 0.169 \text{ft} \]

\[ \frac{L}{a} = \frac{9.5 \text{ft}}{4.581 \text{ft}} = 2.375 \text{ft} > e \quad \text{o.k.} \]

**Sliding**

\[ F.S. = \frac{1.003k + 12.111k \left( \frac{9.5}{22} \text{ft} \right) \tan 24^\circ + \left( \frac{24}{22} \text{ft} \right) \tan \left( \frac{24^\circ}{2} \right)}{4.204k} = 1.161 > 1.125 \quad \text{o.k.} \]

**Footing Pressure**

for \( e \leq \frac{L}{a} \):

\[ P = \sum \frac{V}{L} \left[ 1 + \frac{6e}{L} \right] \]

\[ P_H = \text{pressure at heel} \]

\[ P_T = \text{pressure at toe} \]

\[ P_H = \frac{12.111k}{9.5 \text{ft}} \left( 1 - \frac{6(0.169 \text{ft})}{9.5 \text{ft}} \right) = 1.139 \frac{k}{\text{ft}^2} \]

\[ P_T = \frac{12.111k}{9.5 \text{ft}} \left( 1 + \frac{6(0.169 \text{ft})}{9.5 \text{ft}} \right) = 1.411 \frac{k}{\text{ft}^2} \]

Allowable soil pressure for earthquake = 2(allowable soil pressure)

\[ (2) \frac{4k}{\text{ft}^2} = \frac{8k}{\text{ft}^2} > 1.411 \frac{k}{\text{ft}^2} \quad \text{o.k.} \]

**Reinforcement-Stem**

\[ d = 11" - 2" - (1/2)(0.5") = 8.75" \]

\[ b = 12" \]

\[ f_c' = 3,000 \text{ psi} \]

**Without Earthquake**

\[ P_{AH} = \frac{1}{2} \left[ 0.120 \frac{k}{\text{ft}} \left( 0.546 \right) \left( 6.944 \text{ft} \right)^2 \left( 1 \text{ft} \cos 18.435^\circ \right) \right] = 1.499k \]

\[ \gamma = 1.3 \]

\[ \beta_E = 1.3 \quad \text{(active lateral earth pressure)} \]

\[ M_u = (1.3)(1.3)(1.499k)(2.315ft) = 5.865 \text{ (ft-k)} \]

**With Earthquake**

\[ k_h = 0.05 \]

\[ k_v = 0 \]
\( \theta = 2.862^\circ \)

1992 AASHTO Div. IA Commentary

\[ \delta = \frac{\phi}{2} = \frac{24^\circ}{2} = 12^\circ \] for angle of friction between soil and wall. This criteria is used only for seismic loading if the angle of friction is not known.

\( \phi = 24^\circ \)

\( i = 18.435^\circ \)

\( \beta = 0^\circ \)

\( K_{AE} = 0.654 \)

\[ P_{AEH} = \frac{1}{2} \gamma_s K_{AE} H^2 \cos \delta \]

\[ P_{AEH} = \frac{1}{2} [0.654/1000 lb/ft^2] (6.944 ft)^2 (1 ft \cos 12^\circ) = 1.851k \]

\[ M_u = (1.499k)(2.315 ft) + (1.851k - 1.499k)(0.6(6.944 ft)) = 4.936(ft - k) \]

The moment without earthquake controls.

\[ R_n = \frac{M_u}{bd} = \frac{5.866(ft-k)}{0.9/1/ft}(8.75 in)^2 \left( \frac{1000 ft}{k} \right) = 85.116 psi \]

\[ \rho = \frac{0.85/1/k}{f_r} \left[ 1 - \frac{1 - 2R_n}{0.85k} \right] \]

\[ \rho = \frac{0.85/3,000 psi}{60,000 psi} \left[ 1 - \frac{1 - 2(85.116 psi)}{0.85/3,000 psi} \right] = 0.00144 \]

AASHTO 8.17.1.1 & 8.15.2.1.1

\[ \rho_{min} = 1.7 \left[ \frac{h}{d} \right]^{1/2} \left[ \frac{f_c}{f_r} \right] = 1.7 \left[ \frac{11 ft}{8.75 in} \right]^{1/2} \left[ \frac{3,000 psi}{80,000 psi} \right] = 0.00245 \]

Use \( \rho = \frac{4}{3} \rho_{min} = \frac{4}{3}(0.00144) = 0.00192 \)

\[ A_{Shw} = \rho bd = 0.00192(12 in)(8.75 in) = 0.202 in^2 \]

One # 4 bar has \( A_b = 0.196 in^2 \)

\[ \frac{s}{0.196 in^2} = \frac{12 in}{0.0222 in^2} \]

\[ s = 11.64 in \]

Use # 4's @ 10" cts.

Check Shear

\( V_u \geq \phi V_n \)

Without Earthquake

\[ V_u = (1.3)(1.3)(1.499k) = 2.533k \]

With Earthquake

\[ V_u = 1.851k \]

The shear force without earthquake controls.

\[ V_u = \frac{2.533k}{0.85/12in/ft}(1000 ft/k) = 28.4 psi \]

\[ V_c = 2 \sqrt{3000 psi} = 109.5 psi > 28.4 psi \quad \text{o.k.} \]
Reinforcement-Footing-Heel

Note: Earthquake will not control and will not be checked.

\[ \beta_E = 1.0 \quad \text{(vertical earth pressure)} \]

\[ d = 18'' - 3'' - (1/2)(0.750'') = 14.625'' \]

\[ b = 12'' \]

\[ f_c' = 3,000 \text{ psi} \]

\[ M_u = 1.3[5.556k + 1.500k(3.333ft)] + 0.889k(4.444ft) + 1.178k(6.667ft)] \]

\[ M_u = 45.919(ft-k) \]

\[ R_n = \frac{0.9(1ft/(14.625in)^2)(1000\beta)}{k} = 238.5\text{psi} \]

\[ \rho = \frac{0.85(3,000\text{psi})}{60,000\text{psi}} \left[ 1 - \sqrt{1 - \frac{2(238.5\text{psi})}{0.85(3,000\text{psi})}} \right] = 0.00418 \]

\[ \rho_{min} = 1.7 \left[ \frac{18\text{in}}{14.625\text{in}} \right]^2 \left[ \frac{3,000\text{psi}}{60,000\text{psi}} \right] = 0.00235 \]

\[ A_{Sreq} = 0.00418(12\text{in})(14.625\text{in}) = 0.734\text{in}^2 \]

Use # 6's @ 7" cts.

Check Shear

Shear shall be checked at back face of stem.

\[ V_u = 1.3[5.556k + 1.500k + 0.889k + 1.178k] = 11.860k \]

\[ V_u = \frac{11.860k}{0.85(12\text{in})(14.625\text{in})} = 79.5\text{psi} < 2\sqrt{3,000\text{psi}} = 109.5\text{psi} \quad \text{o.k.} \]

Reinforcement-Footing-Toe
Without Earthquake

Apply Load Factors

load 4 (weight) = 0.431k(1.3)(1.0) = 0.560k

$\beta_E = 1.3$ for lateral earth pressure for retaining walls.

$\beta_E = 1.0$ for vertical earth pressure.

$\Sigma M_{DT} = 12.567(ft - k)(1.3)(1.3) = 21.238(ft - k)$

$\Sigma M_R = [54.556(ft - k) + 11.192(ft - k)(1.3)(1.0)] = 85.472(ft - k)$

$\Sigma V = 11.417k(1.3)(1.0) = 14.842k$

$x = \frac{85.472(ft-k) - 21.238(ft-k)}{14.842k} = 4.328ft$

$e = \frac{9.5ft}{2} - 4.328ft = 0.422ft$

$P_H = \frac{14.842k}{11ft(9.5ft)} \left[ 1 - \frac{6(0.422ft)}{9.5ft} \right] = 1.146\frac{k}{ft^2}$

$P_T = \frac{14.842k}{11ft(9.5ft)} \left[ 1 + \frac{6(0.422ft)}{9.5ft} \right] = 1.979\frac{k}{ft^2}$

$P = \left[ \frac{1.979\frac{k}{ft} - 1.146\frac{k}{ft}}{9.5ft} \right] (7.583ft) + 1.146\frac{k}{ft} = 1.811\frac{k}{ft}$

$M_u = 1.811\frac{k}{ft} \left( \frac{1.917ft^2}{2} \right) + \frac{1}{2}(1.917ft) \left( 1.979\frac{k}{ft} - 1.811\frac{k}{ft} \right) \frac{2}{3} - 0.560k(0.958ft)$

$M_u = 2.997(ft-k)$

With Earthquake

$P_H = 1.139\frac{k}{ft}$

$P_T = 1.411\frac{k}{ft}$

$P = \left[ \frac{1.411\frac{k}{ft} - 1.139\frac{k}{ft}}{9.5ft} \right] (7.583ft) + 1.139\frac{k}{ft} = 1.356\frac{k}{ft}$

$M_u = 1.356\frac{k}{ft} \left( \frac{1.917ft^2}{2} \right) + \frac{1}{2}(1.917ft) \left( 1.411\frac{k}{ft} - 1.356\frac{k}{ft} \right) \frac{2}{3} - 0.431k(0.958ft)$

$M_u = 2.146(ft-k)$

The moment without earthquake controls.

$R_n = \frac{2.997(ft-k) \times 0.9(1ft/14.0psi)}{0.85(3,000psi)} \times (1000\frac{lb}{ft}) = 16.990psi$

$\rho = \frac{0.85(3,000psi)}{60,000psi} \left[ 1 - \sqrt{1 - \frac{2(16.990psi)}{0.85(3,000psi)}} \right] = 0.000284$
\[ \rho_{min} = 1.7 \left( \frac{18\text{in}}{14.0\text{in}} \right)^2 \frac{3000\text{psi}}{60,000\text{psi}} = 0.00257 \]

Use \( \rho = \frac{4}{3} \rho = \frac{4}{3}(0.000284) = 0.000379 \)

\[ A_{SReq} = 0.000379(12\text{in})(14.0\text{in}) = 0.064 \text{in}^2 \]

\[ \frac{12\text{in}}{0.064\text{in}^2} = \frac{s}{0.196\text{in}^2} \]

\( s = 36.8\text{in} \)

Minimum is # 4 bars at 12 inches. These will be the same bars that are in the back of the stem. Use the smaller of the two spacings.

Use # 4's @ 10" cts.

**Check Shear**

Shear shall be checked at a distance "d" from the face of the stem.

**Without Earthquake**

\[ P_d = \left[ \frac{1.979}{9.5} - \frac{1.146}{9.5} \right] (8.750\text{ft}) + 1.146\frac{k}{k} = 1.913k \]

\[ V_u = \frac{1.979}{2} (0.750\text{ft}) - 1.3 \left[ 0.225\frac{k}{k} \right] (0.750\text{ft}) = 1.240k \]

**With Earthquake**

\[ P_d = \left[ \frac{1.411}{9.5} - \frac{1.139}{9.5} \right] (8.750\text{ft}) + 1.139\frac{k}{k} = 1.390k \]

\[ V_u = \frac{1.411}{2} (0.750\text{ft}) - 0.225\frac{k}{k} (0.750\text{ft}) = 0.788k \]

Shear without earthquake controls.

\[ \frac{V_u}{\phi} = \frac{1.240k}{0.85(12\text{in})(14.0\text{in})} (1000 \frac{\text{lb}}{k}) = 8.7\text{psi} < 2 \sqrt{3,000\text{psi}} = 109.5\text{psi} \quad \text{o.k.} \]

**Reinforcement-Shear Key**

* Include 1’ of eroded fill

*3.379 k

The passive pressure is higher without earthquake loads.

\( \gamma = 1.3 \)

\( \beta_E = 1.3 \) (lateral earth pressure)
d = 12" - 3" - (1/2)(0.5") = 8.75"

b = 12"

\[ M_u = (3.379k)(1.360ft)(1.3)(1.3) = 7.764(ft - k) \]

\[ R_n = \frac{7.764(ft - k)}{0.9(1/8.75in)^2}(1000\frac{lb}{k}) = 112.677\text{psi} \]

\[ \rho = \frac{0.85(3000\text{psi})}{60000\text{psi}} \left[ 1 - \sqrt{1 - \frac{2(112.677\text{psi})}{0.85(3000\text{psi})}} \right] = 0.00192 \]

\[ \rho_{min} = 1.7 \frac{12\text{in}}{8.75\text{in}} \gamma^2 \frac{3000\text{psi}}{60000\text{psi}} = 0.00292 \]

Use \[ \rho = \frac{4}{3} \rho_{min} = \frac{4}{3} (0.00192) = 0.00256 \]

\[ A_{S_{req}} = 0.00256(12\text{in})(8.75\text{in}) = 0.269\frac{\text{in}^2}{\text{ft}} \]

Use # 4 @ 8.5 in cts.

Check Shear

\[ \frac{\nu}{\phi} = \frac{1.3(3.379k)(1.3)}{0.85(12\text{in})(8.75\text{in})}(1000\frac{lb}{k}) = 64.0\text{psi} < 2\sqrt{3000\text{psi}} = 109.5\text{psi} \quad \text{o.k.} \]

**Reinforcement Summary**

- 3" Cl.
- 2" Cl.
- 4" Cl.
- 3" Cl.
- # 4 @ 12" cts.
- # 4 @ 18" cts.
- # 4 @ 10" cts.
- # 4 @ 12" cts.
- # 4 @ 12" cts.
- # 6 @ 7" cts.
- # 4 @ 18" cts.
- # 4 @ 8.5" cts.
- # 4 @ 18" cts.
- # 4 @ 8.5" cts.
- # 4 @ 8.5" cts.
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- # 4 @ 8.5" cts.
- # 4 @ 8.5" cts.
3.4 Example 2: L-Shaped Cantilever Wall

\[ f'_c = 4,000 \text{ psi} \quad f_y = 60,000 \text{ psi} \quad \phi = 29^\circ \quad \gamma_s = 120 \text{pcf} \]

Allowable soil pressure = 1.5 tsf = 3.0 ksf

Retaining wall is located in Seismic Performance Category (SPC) A.

\[ \delta = \tan^{-1} \frac{1}{2.5} = 21.801^\circ \]

\[ C_a = \cos \delta \left[ \frac{\cos \phi}{\cos \phi + \cos^2 \delta} \right] = 0.462 \]

\[ C_p = \tan^2 \left[ 45 + \frac{\phi}{2} \right] = 2.882 \]

\[ P_A = \frac{1}{2} \gamma_s C_a H^2 = \frac{1}{2} (0.120 \frac{k}{ft})(0.462)(4.958ft)^2 = 0.681k \]

For sliding, \( P_p \) is assumed to act only on the portion of key below the frost line which is set at an 18in. depth on the southern border.

\[ P_p = \frac{1}{2} (0.120 \frac{k}{ft})(2.882)(2.458ft)^2 - (1.500ft)^2 = 0.656k \]

**Assumptions**

- Design is for a unit length (1 foot) of wall.
- Sum moments about the toe at the bottom of the footing for overturning.
- F.S. for overturning \( \geq 2.0 \) for footings on soil.
- F.S. for sliding \( \geq 1.5 \) for footings on soil.
- Resultant of dead load and earth pressure to be in back half of the middle third of the footing if subjected to frost heave.
- For all loading combinations the resultant must be in the middle third of the footing except for collision loads.
- The top 12 inches of the soil is not neglected in determining the passive pressure because the soil there will be maintained.
Frost line is set at 18 inches at the south border for Missouri.

Portions of shear key which are above the frost line are assumed not to resist sliding by passive pressure.

Use of a shear key shifts the failure plane to "B" where resistance to sliding is also provided by friction of soil along the failure plane in front of the shear key. Friction between the soil and concrete behind the shear key will be neglected.

Soil cohesion along the failure plane is neglected.

Live loads can move to within 1 foot of the stem face and 1 foot from the toe.

The wall is designed as a cantilever supported by the footing.

Footing is designed as a cantilever supported by the wall. Critical sections for bending and shear will be taken at the face of the wall.

Load factors for AASHTO Groups I-VI for design of concrete are:

- $\gamma = 1.3$
- $\beta_E = 1.3$ for horizontal earth pressure on retaining walls.
- $\beta_E = 1.0$ for vertical earth pressure.
- $\beta_{LL} = 1.67$ for live loads and collision loads.

### Dead Load and Earth Pressure - Stability and Pressure Checks

<table>
<thead>
<tr>
<th>Load</th>
<th>Force (k)</th>
<th>Arm (in)</th>
<th>Moment (ft-k)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>(0.833ft)(5.167ft)(0.150k/ft$^3$) = 0.646</td>
<td>5.333</td>
<td>3.444</td>
</tr>
<tr>
<td>2</td>
<td>(0.958ft)(5.750ft)(0.150k/ft$^3$) = 0.827</td>
<td>2.875</td>
<td>2.376</td>
</tr>
<tr>
<td>3</td>
<td>(1.000ft)(1.500ft)(0.150k/ft$^3$) = 0.225</td>
<td>4.250</td>
<td>0.956</td>
</tr>
</tbody>
</table>

$\sum V = 1.698$  \hspace{1cm} $\sum M_R = 6.776$

| $P_{AV}$ | 0.253 | 5.750 | 1.455 |
| $\sum V = 1.951$ | $\sum M_R = 8.231$

| $P_{AH}$ | 0.633 | 1.653 | 1.045 |
| $P_p$ | 0.656 | 1.06a |

$\sum M_{OT} = 1.045$

a. The passive pressure at the shear key is ignored in overturning checks.

**Overturning**

$$F.S. = \frac{\sum M_R}{\sum M_{OT}} = \frac{8.231(ft-k)}{1.045(ft-k)} = 7.877 \geq 2.0 \hspace{1cm} \text{o.k.}$$

**Location of Resultant**

MoDOT policy is that the resultant must be in the back half of the middle third of the footing when considering dead and earth loads:

$$\frac{5.750ft}{2} = 2.875ft \leq \bar{x} \leq \left( \frac{5.750ft}{2} + \frac{5.750ft}{6} \right) = 3.833ft$$

$$\bar{x} = \frac{M_{NET}}{\sum V} = \frac{8.231(ft-k)}{1.045(ft-k)} = 3.683ft \hspace{1cm} \text{o.k.}$$

**Sliding**
Footing Pressure

\[ F.S. = \frac{P_{F} \sum V \left( \frac{L_2}{G_e} \tan \phi_{s-s} + \frac{L_3}{G_f} \tan \phi_{s-c} \right)}{P_{FH}} \]

where: \( \phi_{s-s} \) = angle of internal friction of soil
\( \phi_{s-c} \) = angle of friction between soil and concrete = \((2/3)\phi_{s-s}\)

\[ F.S. = \frac{0.656k + \left(1.951k\right) \left(\frac{3.75}{5.75}\right) \tan 29^\circ + \left(\frac{18}{5.75}\right) \tan \left(\frac{3}{5} \tan 29^\circ\right)}{0.633k} = 2.339 \geq 1.5 \text{ o.k.} \]

Dead Load, Earth Pressure, and Live Load - Stability and Pressure Checks

Stability is not an issue because the live load resists overturning and increases the sliding friction force.

The live load will be distributed as:
\[ F_{LL} = \frac{LLWL}{E} \text{ where } E = 0.8X + 3.75 \]

\[ X = \text{distance in feet from the load to the front face of wall} \]

The live load will be positioned as shown by the dashed lines above. The bearing pressure and resultant location will be determined for these two positions.

Live Load 1 ft From Stem Face
RESULTANT ECCENTRICITY

X = 1 ft
E = 0.8(1 ft) + 3.75 = 4.55 ft

\[ F_{LL} = \frac{16k}{4.55 ft} (1 ft) = 3.516 k \]

\[ x = \frac{M_{net}}{\sum V} = \frac{8.231(ft-k)+3.516k(3.917ft-1.045ft-k)}{1.951k+3.516k} = 3.834 ft \]

\[ e = x - \frac{L}{2} = 3.834 ft - \frac{5.75 ft}{2} = 0.959 ft \leq \frac{L}{6} = \frac{5.75 ft}{6} = 0.958 ft \quad \text{o.k.} \]

FOOTING PRESSURE

\[ P = \sum \frac{Vr}{L} \left[ 1 \pm \frac{6e}{L} \right] \]

Allowable Pressure = 3.0 ksf

Heel: \( P_H = \frac{5.467k}{(1 ft)(5.75 ft)} \left[ 1 + \frac{6(0.959 ft)}{5.75 ft} \right] = 1.902 ksf \quad \text{o.k.} \)

Toe: \( P_T = \frac{5.467k}{(1 ft)(5.75 ft)} \left[ 1 - \frac{6(0.959 ft)}{5.75 ft} \right] = 0.000 ksf \quad \text{o.k.} \)

LIVE LOAD 1 ft FROM TOE

RESULTANT ECCENTRICITY

X = 3.917 ft
E = 0.8(3.917 ft) + 3.75 = 6.883 ft

\[ F_{LL} = \frac{16k}{6.883 ft} (1 ft) = 2.324 k \]

\[ x = \frac{8.231(ft-k)+2.324k(1 ft)-1.045(ft-k)}{1.951k+2.324k} = 2.225 ft \]

\[ e = \frac{L}{2} - x = \frac{5.75 ft}{2} - 2.225 ft = 0.650 ft \leq \frac{L}{6} = \frac{5.75 ft}{6} = 0.958 ft \quad \text{o.k.} \]

FOOTING PRESSURE

Allowable Pressure = 3.0 ksf

Heel: \( P_H = \frac{4.275k}{(1 ft)(5.75 ft)} \left[ 1 - \frac{6(0.650 ft)}{5.75 ft} \right] = 0.239 ksf \quad \text{o.k.} \)

Toe: \( P_H = \frac{4.275k}{(1 ft)(5.75 ft)} \left[ 1 + \frac{6(0.650 ft)}{5.75 ft} \right] = 1.248 ksf \quad \text{o.k.} \)

DEAD LOAD, EARTH PRESSURE, COLLISION LOAD, AND LIVE LOAD - STABILITY AND PRESSURE CHECKS

Revised: December 1998
During a collision, the live load will be close to the wall so check this combination when the live load is one foot from the face of the stem. Sliding (in either direction) will not be an issue. Stability about the heel should be checked although it is unlikely to be a problem. There are no criteria for the location of the resultant, so long as the footing pressure does not exceed 125% of the allowable. It is assumed that the distributed collision force will develop an equal and opposite force on the fillface of the back wall unless it exceeds the passive pressure that can be developed by soil behind the wall.

\[ F_{LL} = 3.516k \]

\[ F_{COLL} = \frac{10k}{2(3ft)} (1ft) = 1.667k \]

\[ C_P = \cos \delta \left( \frac{\cos \delta + \cos^2 \delta \cos^2 \phi}{\cos \delta - \cos^2 \delta \cos^2 \phi} \right) = 1.867 \]

\[ P_{PH} = \frac{1}{2} \gamma s C_P H^2 \cos \delta = \frac{1}{2} (0.120kcf)(1.867)(4.958ft)^2 \cos(21.801^\circ) \]

\[ P_{PH} = 2.556k > F_{COLL}, \text{ Thus the soil will develop an equal but opp. force.} \]

**Overturning About the Heel**

\[ F.S. = \frac{(0.646k)(0.417ft)+(0.827k)(2.875ft)+(0.225k)(1.500ft)+(3.516k)(1.833ft)+(1.667k)(4.958ft)}{(1.667k)(3.958ft)} = 4.958ft \]

\[ F.S. = \frac{12.184(ft-k)}{6.598(ft-k)} = 1.847 \geq 1.2 \quad \text{O.K.} \]

**Footing Pressure**

\[ \chi = \frac{12.184(ft-k)-6.598(ft-k)}{1.951k+3.516k} = 1.022ft \text{ from heel} \]

\[ e = \frac{5.75ft}{2} - 1.022ft = 1.853ft \]

Allowable Pressure = (1.25)(3.0ksf) = 3.75ksf
Heel: \[ P_H = \frac{2 \sum v_i}{3d} = \frac{2(5.467k)}{3(7.75) - 1.853ft} = 3.566ksf \] o.k.

**Stem Design-Steel in Rear Face**

\[ \gamma = 1.3 \]
\[ \beta_E = 1.3 \text{ (active lateral earth pressure)} \]
\[ d = 10\text{in} - 2\text{in} - \frac{0.5\text{in}}{2} = 7.75\text{in} \]

\[ P_{AH} = \frac{1}{2} \gamma s C_a H^2 \cos \delta = \frac{1}{2} (0.120 \frac{k}{H}) (0.462)(4\text{ft})^2(1\text{ft})\cos 21.801^\circ \]

\[ P_{AH} = 0.412k \]

\[ M_u = (1.333\text{ft})(0.412k)(1.3)(1.3) = 0.928(ft - k) \]

\[ R_n = \frac{M_u}{abd} = \frac{0.928(ft - k)}{(0.9)(1\text{ft})(7.75\text{in})^2} \left( 1000 \frac{\text{lb}}{k} \right) = 17.160\text{psi} \]

\[ \rho = \frac{0.85f_c}{f_y} \left[ 1 - \sqrt{ \frac{1 - \frac{2R_n}{f_y}}{0.85f_c} } \right] \]

\[ \rho = \frac{0.85(4,000\text{psi})}{60,000\text{psi}} \left[ 1 - \sqrt{ \frac{1 - \frac{2(17.160\text{psi})}{0.85(4000\text{psi})}}{0.85(4000\text{psi})} } \right] = 0.000287 \]

\[ \rho_{min} = 1.7 \left( \frac{h}{d} \right)^2 \frac{\sqrt{c}}{f_y} \]

\[ \rho_{min} = 1.7 \left( \frac{10\text{in}}{7.75\text{in}} \right)^2 \frac{\sqrt{4,000\text{psi}}}{60,000\text{psi}} = 0.00298 \]

Use \[ \rho = \frac{4}{3} \rho = \frac{4}{3}(0.000287) = 0.000382 \]

\[ A_{sreq} = \rho bd = 0.000382(12\text{in})(7.75\text{in}) = 0.036\text{in}^2 \]

One # 4 bar has \[ A_g = 0.196\text{in}^2 \] so the required minimum of one # 4 bar every 12 in. controls.

Use # 4's @ 12 in. (min)
(These bars are also the bars in the bottom of the footing so the smaller of the two required spacings will be used.)

**Check Shear**

\[
\frac{V_u}{\phi} \leq V_n
\]

\[
\frac{V_u}{\phi} = \frac{(1.3)(1.3)(0.412k)}{0.85(12in/7.75in)}(1000lb/k) = 8.8\text{psi}
\]

\[
\nu_c = 2\sqrt{\frac{k}{c}}
\]

\[
\nu_c = 2\sqrt{4,000\text{psi}} = 126.5\text{psi} > 8.8\text{psi} \quad \text{o.k.}
\]

**Stem Design-Steel in Front Face (Collision Loads)**

![Diagram of a stem design with forces and dimensions]

The soil pressure on the back of the stem becomes passive soil pressure during a collision, however this pressure is ignored for reinforcement design.

\[
\gamma = 1.3
\]

\[
\beta_{LL} = 1.67
\]

\[
d = 10in - 1.5in - 0.5in - \frac{0.5in}{2} = 7.75in
\]

\[
F_{COLL} = \frac{10k}{2L} = \frac{10k}{(2)(3ft)} = 1.667\frac{k}{ft}
\]

\[
M_u = 1.667\frac{k}{ft}(1ft)(3ft)(1.3)(1.67) = 10.855(ft-k)
\]

\[
R_n = \frac{10.855(ft-k)}{0.9(1ft/7.75in^2)}(1000lb/k) = 200.809\text{psi}
\]

\[
\rho = \frac{0.85(4,000\text{psi})}{60,000\text{psi}} \left[1 - \sqrt{1 - \frac{2(200.809\text{psi})}{0.85(4,000\text{psi})}}\right] = 0.00345
\]

\[
\rho_{min} = 1.7\left[\frac{10in}{7.75in}\right]^2\frac{4,000\text{psi}}{60,000\text{psi}} = 0.00298
\]

\[
A_{S_{req}} = 0.00345(12in)(7.75in) = 0.321\text{in}^2
\]

One # 4 bar has \(A_s = 0.196\text{in}^2\)

\[
\frac{s}{0.196\text{in}^2} = \frac{12in}{0.321\text{in}^2}
\]
s = 7.3in

Use # 4's @ 7 in.

Check Shear

\[ \frac{v}{\phi} \frac{(1.3)(1.67)(1.667k)}{(0.85)(12in)(7.75in)} \frac{(1000 \text{lb})}{k} = 45.8 \text{psi} < 126.5 \text{psi} \quad \text{o.k.} \]

Footing Design - Bottom Steel

It is not considered necessary to design footing reinforcement based upon a load case which includes collision loads.

Dead Load and Earth Pressure Only

\[ \text{Footing wt.} = \left[ \frac{11.5}{12} \text{ft} \right] \left( 4.917 \text{ft} \right) \left( 0.150 \frac{k}{\text{ft}^2} \right) \left( 1 \text{ft} \right) = 0.707k \]

\[ \beta_E = 1.3 \quad \text{(lateral earth pressure)} \]

\[ \gamma = 1.3 \]

Apply Load Factors:

\[ \Sigma V = 1.951k(1.3) = 2.536k \]

\[ \Sigma M_R = 8.231(\text{ft} - k)(1.3) = 10.700(\text{ft} - k) \]

\[ \Sigma M_{OT} = 1.045(\text{ft} - k)(1.3)(1.3) = 1.766(\text{ft} - k) \]

\[ \text{Footing wt.} = 0.707k(1.3) = 0.919k \]

\[ x = \frac{10.700(\text{ft} - k) - 1.766(\text{ft} - k)}{2.536k} = 3.523\text{ft} \]

\[ e = 3.523\text{ft} - \frac{5.75\text{ft}}{2} = 0.648\text{ft} \]

\[ P_H = \frac{2.536k}{(1\text{ft})(5.75\text{ft})} \left[ 1 + \frac{6(0.648\text{ft})}{5.75\text{ft}} \right] = 0.739\text{ksf} \]

\[ P_T = \frac{2.536k}{(1\text{ft})(5.75\text{ft})} \left[ 1 - \frac{6(0.648\text{ft})}{5.75\text{ft}} \right] = 0.143\text{ksf} \]

\[ P_W = 0.143\text{ksf} + [0.739\text{ksf} - 0.143\text{ksf}] \left[ \frac{4.917\text{ft}}{5.75\text{ft}} \right] = 0.653\text{ksf} \]

Moment at Wall Face:
Dead Load, Earth Pressure, and Live Load

**Live Load 1 ft From Stem Face**

![Diagram of live load calculation](image)

\[ \beta_E = 1.3 \] (lateral earth pressure)
\[ \beta_{LL} = 1.67 \]
\[ \gamma = 1.3 \]

Apply Load Factors:
\[ F_{LL} = 3.516k(1.3)(1.67) = 7.633k \]
\[ \Sigma V = 7.633k + 1.951k(1.3) = 10.169k \]
\[ \Sigma M_{OT} = 1.045(ft - k)(1.3)(1.3) = 1.766(ft - k) \]
\[ \Sigma M_R = 8.231(ft - k)(1.3) + 3.917ft(7.633k) = 40.599(ft - k) \]
\[ x = \frac{40.599(ft-k) - 1.766(ft-k)}{10.169k} = 3.819\text{ft} \]
\[ e = 3.819\text{ft} - \frac{5.75\text{ft}}{2} = 0.944\text{ft} \]
\[ P_T = \left\lceil \frac{10.169k}{(1\text{ft})(5.75\text{ft})} \right\rceil \left[ 1 - \frac{6(0.944\text{ft})}{5.75\text{ft}} \right] = 0.026k\text{sf} \]
\[ P_H = \left\lceil \frac{10.169k}{(1\text{ft})(5.75\text{ft})} \right\rceil \left[ 1 + \frac{6(0.944\text{ft})}{5.75\text{ft}} \right] = 3.511k\text{sf} \]
\[ P_W = 0.026k\text{sf} + [3.511k\text{sf} - 0.026k\text{sf}] \left\lceil \frac{4.917\text{ft}}{5.75\text{ft}} \right\rceil = 3.006k\text{sf} \]
\[ P_{LL} = 0.026k\text{sf} + [3.511k\text{sf} - 0.026k\text{sf}] \left\lceil \frac{3.917\text{ft}}{5.75\text{ft}} \right\rceil = 2.400k\text{sf} \]

Footing wt. from face of wall to toe:
\[ \text{Footing wt.} = 1.3\left\lceil \frac{11.5\text{ft}}{12\text{ft}} \right\rceil \left[ 4.917\text{ft} \right] \left\lceil 0.150\text{ft} \right\rceil (1\text{ft}) = 0.919k \]

Footing wt. from LLWL to toe:
Footing wt. = 1.3[11.5 ft/(3.917 ft)]0.150 k ft

Moment at Wall Face:
\[ M_W = 0.026 \frac{k}{\pi} \left( \frac{4.917}{2} \right)^2 - 7.633k(1ft) \]
\[ + \frac{1}{2} \left[ 3.006 \frac{k}{\pi} - 0.026 \frac{k}{\pi} \left( \frac{4.917}{2} \right)^2 \right] - 0.919k \left( \frac{4.917}{2} \right) \]
\[ M_W = 2.430(ft - k) \]

Moment at LLWL
\[ M_{LL} = 0.026 \frac{k}{\pi} \left( \frac{3.917}{2} \right)^2 - 0.732k \left( \frac{3.917}{2} \right) \]
\[ + \frac{1}{2} \left[ 2.400 \frac{k}{\pi} - 0.026 \frac{k}{\pi} \left( \frac{3.917}{2} \right)^2 \right] = 4.837(ft - k) \]

**Live Load 1 ft From Toe**

![Diagram of live load](image)

Apply Load Factors:
\[ F_{LL} = 2.324(1.3)(1.67) = 5.045k \]
\[ \Sigma V = 5.045k + 1.951k(1.3) = 7.581k \]
\[ \Sigma M_{OT} = 1.045(ft - k)(1.3)(1.3) = 1.766(ft - k) \]
\[ \Sigma M_R = 8.231(ft - k)(1.3) + 5.045k(1ft) = 15.745(ft - k) \]

\[ x = \frac{15.745(ft - k) - 1.766(ft - k)}{7.581k} = 1.844ft \]
\[ e = \frac{5.75ft}{2} - 1.844ft = 1.031ft \]
\[ P_H = 0ksf \]
\[ P_T = \frac{2(7.581k)}{3(1ft)} \frac{5.75ft}{2} - 1.031ft = 2.741ksf \]
\[ L_1 = 3 \left[ \frac{L}{2} - e \right] \]
\[ L_1 = 3 \left[ \frac{5.75ft}{2} - 1.031ft \right] = 5.532ft \]
\[ P_W = 2.741ksf \left[ \frac{0.616ft}{5.532ft} \right] = 0.305ksf \]
\[ P_{LL} = 2.741ksf \left[ \frac{4.432ft}{5.532ft} \right] = 2.196ksf \]
Moment at Wall Face:
\[ M_W = -5.045k(3.917\text{ ft}) - 0.919k \left( \frac{4.917\text{ ft}}{2} \right) + \frac{1}{2} \left( 0.305k \frac{4.917\text{ ft}}{9}\right)^2 (1\text{ ft})^2 = 1.298(\text{ft} - k) \]

Moment at LLWL:
\[ M_{LL} = -0.187k(0.5\text{ ft}) + 2.196k \left( \frac{1\text{ ft}}{2} \right)^2 + \frac{1}{2} \left( \frac{1\text{ ft}}{2} \right)^2 \left( 2.741k \frac{1\text{ ft}}{9}\right)^2 (1\text{ ft}) = 1.186(\text{ft} - k) \]

**Design Flexural Steel in Bottom of Footing**

\[ d = 11.5in - 4in = 7.500in \]

\[ M_d = 4.837(\text{ft} - k) \quad \text{(controlling moment)} \]

\[ R_n = \frac{4.837(\text{ft} - k)}{0.9(1\text{ft/7.5in})^2} = 0.096\text{ksi} \]

\[ \rho = \frac{0.05(4,000\text{ psi})}{60,000\text{ psi}} \left[1 - \sqrt{1 - \frac{20(0.096\text{ksi})}{0.85(4\text{ksi})}}\right] = 0.00162 \]

\[ \rho_{\text{min}} = 1.7 \left[ \frac{11.5in}{7.5in} \right]^2 \frac{4,000\text{psi}}{60,000\text{psi}} = 0.00421 \]

Use \( \rho = \frac{4}{3} \rho = \frac{4}{3}(0.00162) = 0.00216 \)

\[ A_{S_{\text{req}}} = 0.00216(12\text{in})(7.5\text{in}) = 0.194\text{in}^2 \]

\[ \frac{s}{0.196\text{in}^2} = \frac{12\text{in}}{0.194\text{in}^2} \]

\[ s = 12.1\text{in} \]

Use \# 4's @ 12 in. cts. (Also use this spacing in the back of the stem.)

**Check Shear**

**Dead Load and Earth Pressure Only**

\[ V_W = 0.143k \frac{4.917\text{ ft}}{9} + \frac{1}{2} \left( 4.917\text{ ft} \right) 0.653k \frac{4.917\text{ ft}}{9} - 0.919k \]

\[ V_W = 1.038k \]

**Live Load 1 ft From Stem Face**

Shear at the wall can be neglected for this loading case.

\[ V_{LL} = 0.026k \frac{3.917\text{ ft}}{9} + \frac{1}{2} \left( 3.917\text{ ft} \right) 2.400k \frac{3.917\text{ ft}}{9} - 0.026k \]

\[ V_{LL} = 4.019k \]
Live Load 1 ft From Toe

\[
V_W = 0.305 \frac{k}{ft} (4.917 ft) + \frac{1}{2} (4.917 ft) \left( 2.741 \frac{k}{ft} - 0.305 \frac{k}{ft} \right) - 0.919 k - 5.045 k
\]

\[
V_W = 1.525 k
\]

\[
V_{LL} = 2.196 \frac{k}{(1 ft)} + \frac{1}{2} (1 ft) \left( 2.741 \frac{k}{ft} - 2.196 \frac{k}{ft} \right) - 0.187 k
\]

\[
V_{LL} = 2.282 k
\]

Use \( V_U = 4.019 k \)

\[
\frac{V_U}{V} = \frac{4019 \text{(lbs)}}{0.85(12\text{in})(7.5\text{in})} = 52.5 \text{psi} < 2 \sqrt{4000 \text{psi}} = 126.5 \text{psi} \quad \text{o.k.}
\]

Shear Key Design

For concrete cast against and permanently exposed to earth, minimum cover for reinforcement is 3 inches.

\[
d = 12\text{in} - 3\text{in} - \frac{1}{2} \left( \frac{1}{2} \text{in} \right) = 8.75\text{in}
\]

\[
P_1 = 0.120 \frac{k}{ft} (1\text{ft})(2.882) \left( \frac{11.5}{12} \text{ft} \right) = 0.331 \frac{k}{ft}
\]

\[
P_2 = 0.120 \frac{k}{ft} (1\text{ft})(2.882) \left( \frac{29.5}{12} \text{ft} \right) = 0.850 \frac{k}{ft}
\]

\[
M_u = (1.3)(1.3) \left[ 0.331 \frac{k}{ft} (1.5\text{ft})^2 + \frac{1}{2} (1.5\text{ft}) \left( 0.850 \frac{k}{ft} - 0.331 \frac{k}{ft} \right) \frac{3}{2} (1.5\text{ft}) \right]
\]

\[
M_u = 1.287 (ft - k)
\]

\[
R_n = \frac{1.287 (ft-k)}{0.9(1\text{ft})(8.75\text{in})} = 0.0187 \text{ksi}
\]

\[
\rho = \frac{0.85(4,000 \text{psi})}{60,000 \text{psi}} \left[ 1 - \sqrt{1 - \frac{2(0.0187 \text{ksi})}{0.85(4 \text{ksi})}} \right] = 0.000312
\]

\[
\rho_{\min} = 1.7 \left[ \frac{12\text{in}}{8.75\text{in}} \right]^2 \sqrt{\frac{4,000 \text{psi}}{60,000 \text{psi}}} = 0.00337
\]

Use \( \rho = \frac{4}{3} \rho = \frac{4}{3}(0.000312) = 0.000416 \)

\[
A_{S_{\text{req}}} = 0.000416(12\text{in})(8.75\text{in}) = 0.0437 \text{in}^2
\]

\[
\frac{3}{0.196 \text{in}^2} = \frac{12\text{in}}{0.0437 \text{in}^2}
\]
\( s = 53.8\text{in} \)

Use # 4's @ 18 in. cts. (min)

**Check Shear**

\[ V = 0.886k \]

\[ \frac{V}{\phi} = \frac{(1.3)(1.3)(886\text{lbs})}{0.85(12\text{in})(8.75\text{in})} = 16.8\text{psi} < 126.5\text{psi} \quad \text{O.K.} \]

**Reinforcement Summary**
### 3.5 Example 3: Pile Footing Cantilever Wall

**TYPICAL SECTION THROUGH WALL (PILE FOOTING)**

**PLAN OF FOOTING**

- $f_c = 3,000$ psi
- $f_y = 60,000$ psi
- $\phi = 27^\circ$
- $\gamma_s = 120$ pcf
- Pile type: HP 10 x 42
- Allowable pile bearing = 56 tons
- Pile width = 10 inches
- Toe pile batter = 1:3
- Barrier curb weight = 340 lbs/foot of length
- Barrier curb resultant = 0.375 ft. from its flat back
Assumptions

- Retaining wall is located such that traffic can come within half of the wall height to the plane where earth pressure is applied.
- Reinforcement design is for one foot of wall length.
- Sum moments about the centerline of the toe pile at a distance of 6B (where B is the pile width) below the bottom of the footing for overturning.
- Neglect top one foot of fill over toe in determining soil weight and passive pressure on shear key.
- Neglect all fill over toe in designing stem reinforcement.
- The wall is designed as a cantilever supported by the footing.
- Footing is designed as a cantilever supported by the wall.
- Critical sections for bending are at the front and back faces of the wall.
- Critical sections for shear are at the back face of the wall for the heel and at a distance d (effective depth) from the front face for the toe.
- For load factors for design of concrete, see page 1.2-8 of this section.

\[ C_A = \cos \delta \left[ \frac{\cos \delta - \sqrt{\cos^2 \delta - \cos^2 \phi}}{\cos \delta + \sqrt{\cos^2 \delta - \cos^2 \phi}} \right] \]

\[ \delta = 0, \; \phi = 27^\circ \] so \( C_A \) reduces to:

\[ C_A = \frac{1 - \sin \phi}{1 + \sin \phi} = \frac{1 - \sin 27^\circ}{1 + \sin 27^\circ} = 0.376 \]

\[ C_P = \tan^2 \left[ 45^\circ + \frac{\phi}{2} \right] = \tan^2 \left[ 45^\circ + \frac{27^\circ}{2} \right] = 2.663 \]
Table is for stability check (moments taken about C.L. of toe pile at a depth of 6B below the bottom of the footing).

<table>
<thead>
<tr>
<th>Load</th>
<th>Force (kips/ft)</th>
<th>Arm about C.L. of toe pile at 6B below footing (ft)</th>
<th>Moment (ft-kips) per foot of wall length</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dead Load</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1)</td>
<td>0.340</td>
<td>2.542</td>
<td>0.864</td>
</tr>
<tr>
<td>(2)</td>
<td>(1.333ft)(7.000ft)(0.150k/ft³) = 1.400</td>
<td>2.833</td>
<td>3.966</td>
</tr>
<tr>
<td>(3)</td>
<td>(3.000ft)(8.500ft)(0.150k/ft³) = 3.825</td>
<td>4.417</td>
<td>16.895</td>
</tr>
<tr>
<td>(4)</td>
<td>(1.000ft)(1.750ft)(0.150k/ft³) = 0.263</td>
<td>4.417</td>
<td>1.162</td>
</tr>
<tr>
<td><strong>Σ</strong></td>
<td></td>
<td></td>
<td><strong>ΣM₉ = 22.887</strong></td>
</tr>
<tr>
<td><strong>Earth Load</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(5)</td>
<td>(7.000ft)(5.167ft)(0.120k/ft³) = 4.340</td>
<td>6.083</td>
<td>26.400</td>
</tr>
<tr>
<td>(6)</td>
<td>(2.000ft)(2.000ft)(0.120k/ft³) = 0.480</td>
<td>1.167</td>
<td>0.560</td>
</tr>
<tr>
<td><strong>Σ</strong></td>
<td></td>
<td></td>
<td><strong>ΣM₉ = 26.960</strong></td>
</tr>
<tr>
<td><strong>Live Load Surcharge</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Pₛᵥ</strong></td>
<td>(2.000ft)(5.167ft)(0.120k/ft³) = 1.240</td>
<td>6.083</td>
<td><strong>M₉ = 7.543</strong></td>
</tr>
<tr>
<td><strong>Pₛₜ</strong></td>
<td>(2.000ft)(0.376)(10.000ft)(0.120k/ft³) = 0.902</td>
<td>10.000</td>
<td><strong>M₉ = 9.020</strong></td>
</tr>
<tr>
<td><strong>Earth Pressure</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Pₐ</strong></td>
<td>2.256ₐ</td>
<td>8.333</td>
<td><strong>M₉ = 18.799</strong></td>
</tr>
<tr>
<td><strong>Pₚ</strong></td>
<td>3.285ₖ</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Collision Force</strong></td>
<td><strong>Fₜ₉</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Pₛᵥ</strong></td>
<td>(10.000k)(12(7.000ft)) = 0.714</td>
<td>18.000</td>
<td><strong>M₉ = 12.852</strong></td>
</tr>
<tr>
<td><strong>Heel Pile Tension</strong></td>
<td><strong>Pₛₚ</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Pₛᵥ</strong></td>
<td>(3.000tons)(2k/ton)(1pile)/(12.000ft) = 0.500</td>
<td>7.167</td>
<td><strong>M₉ = 3.584</strong></td>
</tr>
<tr>
<td><strong>Toe Pile Batter</strong></td>
<td><strong>Pₛ₂₉</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Pₛᵥ</strong></td>
<td>5.903ₖ</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Passive Pile Pressure</strong></td>
<td><strong>Pₛₚ</strong></td>
<td>0.832ₖ</td>
<td></td>
</tr>
</tbody>
</table>

a. \[ Pₐ = \frac{1}{2} y_s C_s H^2 = \frac{1}{2} \left[ 0.120 \frac{k}{ft^3} \right] (0.376)(10ft)^2 = 2.256 \frac{k}{ft} \]

b. \[ Pₚ = \frac{1}{2} y_s C_s [H^2 - H_i^2] = \frac{1}{2} \left[ 0.120 \frac{k}{ft^3} \right] (2.663)(6.75ft)^2 - (5ft)^2 = 3.285 \frac{k}{ft} \]

c. \[ Pₛₚ = \left( \frac{56 \text{ tons}}{\text{pile}} \right) \left( \frac{k}{\text{ton}} \right) \left( \frac{2}{\text{piles}} \right) \left( \frac{4in}{\sqrt{(12in)^2 + (4in)^2}} \right) \left( \frac{1}{12 \text{ ft}} \right) = 5.903 \frac{k}{\text{ft}} \]

d. \[ Pₛₚ = \frac{1}{2} (2.663)(5ft)^2 \left( 0.120 \frac{k}{ft^3} \right) (0.833 \text{ ft}) \left( 3 \text{ piles} \right) \left( \frac{1}{12 \text{ ft}} \right) = 0.832 \frac{k}{\text{ft}} \]
Retaining Walls - Section 3.62

Table is for bearing pressure checks (moments taken about C.L of toe pile at the bottom of the footing).

<table>
<thead>
<tr>
<th>Load</th>
<th>Force (kips/ft)</th>
<th>Arm about C.L. of toe pile at bottom of footing (ft)</th>
<th>Moment (ft-kips) per foot of wall length</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(1)</td>
<td>0.340</td>
<td>0.875</td>
<td>0.298</td>
</tr>
<tr>
<td>(2)</td>
<td>(1.333ft)(7.000ft)(0.150k/ft³)=1.400</td>
<td>1.167</td>
<td>1.634</td>
</tr>
<tr>
<td>(3)</td>
<td>(3.000ft)(8.500ft)(0.150k/ft³)=3.825</td>
<td>2.750</td>
<td>10.519</td>
</tr>
<tr>
<td>(4)</td>
<td>(1.000ft)(1.750ft)(0.150k/ft³)=0.263</td>
<td>2.750</td>
<td>0.723</td>
</tr>
<tr>
<td></td>
<td>ΣΣV = 5.828</td>
<td>ΣΜᵣ = 13.174</td>
<td></td>
</tr>
<tr>
<td>Earth Load</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(5)</td>
<td>(7.000ft)(5.167ft)(0.120k/ft³)=4.340</td>
<td>4.417</td>
<td>19.170</td>
</tr>
<tr>
<td>(6)</td>
<td>(2.000ft)(2.000ft)(0.120k/ft³)=0.480</td>
<td>-0.500</td>
<td>-0.240</td>
</tr>
<tr>
<td></td>
<td>ΣΣV = 4.820</td>
<td>ΣΜᵣ = 18.930</td>
<td></td>
</tr>
<tr>
<td>Live Load Surcharge</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P_{SV} )</td>
<td>(2.000ft)(5.167ft)(0.120k/ft³)=1.240</td>
<td>4.417</td>
<td>( M_{OT} = 5.477 )</td>
</tr>
<tr>
<td>( P_{SH} )</td>
<td>(2.000ft)(0.376)(10.000ft)(0.120k/ft³)=0.902</td>
<td>5.000</td>
<td>( M_{OT} = 4.510 )</td>
</tr>
<tr>
<td>Earth Pressure</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( P_A )</td>
<td>2.256</td>
<td>3.333</td>
<td>( M_{OT} = 7.519 )</td>
</tr>
<tr>
<td>( P_P )</td>
<td>3.285</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Collision Force ( F_{COL} )</td>
<td>(10.000k)/(2(7.000ft)) = 0.714</td>
<td>13.000</td>
<td>( M_{OT} = 9.282 )</td>
</tr>
<tr>
<td>Heel Pile Tension ( P_{HV} )</td>
<td>(3.000tons)(2k/ton)(1pile)/(12.000ft) = 0.500</td>
<td>5.500</td>
<td>( M_{R} = 2.750 )</td>
</tr>
<tr>
<td>Toe Pile Batter ( P_{TH} )</td>
<td>5.903</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Passive Pile Pressure ( P_{PW} )</td>
<td>0.832</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Investigate a representative 12 foot strip. This will include one heel pile and two toe piles. The assumption is made that the stiffness of a batter pile in the vertical direction is the same as that of a vertical pile.

Neutral Axis Location = \( \frac{2 \times \text{piles}(1.5\text{ft}) + 1 \times \text{piles}(7\text{ft})}{3 \times \text{piles}} = 3.333\text{ft} \) from the toe.

\[
\begin{align*}
I &= A d^2 \\
\text{For repetitive 12ft strip:} & \\
\text{Total pile area} &= 3A \\
I &= 2A(1.833\text{ft})^2 + A(3.667\text{ft})^2 = 20.167(A)\text{ft}^2 \\
\text{For a 1ft unit strip:} & \\
I &= \frac{20.167A}{12\text{ft}} = 1.681(A)\text{ft}^2 \\
\text{Total pile area} &= (3A/12\text{ft})=0.250A
\end{align*}
\]

**Case I**

F.S. for overturning \( \geq 1.5 \)

F.S. for sliding \( \geq 1.5 \)

**Check Overturning**

Neglect resisting moment due to \( P_{oy} \) for this check.

\[
\begin{align*}
\Sigma M_R &= 22.887(ft - k) + 26.960(ft - k) + 3.584(ft - k) \\
\Sigma M_R &= 53.431(ft - k) \\
\Sigma M_{OT} &= 9.020(ft - k) + 18.799(ft - k) = 27.819(ft - k) \\
F.S._{OT} &= \frac{\Sigma M_R}{\Sigma M_{OT}} = \frac{53.431(ft - k)}{27.819(ft - k)} = 1.921 > 1.5 \quad \text{o.k.}
\end{align*}
\]
Check Pile Bearing

Without $P_{sv}:

$$\Sigma V = 5.828k + 4.820k = 10.648k$$

$$e = \frac{\Sigma M}{\Sigma V} = \frac{13.174 + 18.930}{10.648k} = 1.885\text{ft}$$

Moment arm = 1.885ft - 1.833ft = 0.052ft

$$P_T = \frac{\Sigma V}{A} - \frac{Mc}{I} = \frac{10.648k}{0.250A} - \frac{10.648k(0.052ft)(1.833ft)}{1.681(A)/ft^2}$$

$$P_T = 41.988k$$

$$P_H = \frac{10.648k}{A} + \frac{10.648k(0.052ft)(3.667ft)}{1.681(A)/ft^2}$$

$$P_H = 43.800k$$

Allowable pile load = 56tons/pile. Each pile has area $A$ so:

$$P_T = 41.988k \text{ tons/pile} \quad \text{o.k.}$$

$$P_H = 43.800k \text{ tons/pile} \quad \text{o.k.}$$

With $P_{sv}:

$$\Sigma V = 5.828k + 4.820k + 1.240k = 11.888k$$

$$e = \frac{13.174 + 18.930 + 5.477}{11.888k} = 2.149\text{ft}$$

Moment arm = 2.149ft - 1.833ft = 0.316ft

$$P_T = \frac{11.888k}{0.250A} + \frac{11.888k(0.316ft)(1.833ft)}{1.681(A)/ft^2} = 43.456k = 21.728\text{ tons/pile} \quad \text{o.k.}$$

$$P_H = \frac{11.888k}{0.250A} + \frac{11.888k(0.316ft)(3.667ft)}{1.681(A)/ft^2} = 55.747k = 27.874\text{ tons/pile} \quad \text{o.k.}$$

Check Sliding

$$F.S. \text{ Sliding} = \frac{3.285k + 5.903k + 0.832k}{0.902k + 2.256k} = 3.173 \geq 1.5 \quad \text{ o.k.}$$

Case II

F.S. for overturning $\geq 1.2$

F.S. for sliding $\geq 1.2$

Check Overturning

$$\Sigma M_R = (22.887 + 26.960 + 7.543 + 3.584)(ft - k) = 60.974(ft - k)$$

$$\Sigma M_{OT} = (9.020 + 18.799 + 12.852)(ft - k) = 40.671(ft - k)$$

$$F.S. \text{ OT} = \frac{\Sigma M_R}{\Sigma M_{OT}} = \frac{60.974(ft - k)}{40.671(ft - k)} = 1.499 \geq 1.2 \quad \text{o.k.}$$
Check Pile Bearing

\[ e = \frac{\sum M}{\sum V} = \frac{(13.174+18.930+5.477)(ft-k)-(4.510+7.519+9.282)(ft-k)}{(5.828+4.820+1.240)k} = 1.369ft \]

Moment arm = 1.833ft - 1.369ft = 0.464ft

\[ P_T = \sum \frac{V}{A} + \frac{Mc}{I} = \frac{11.888k}{0.250A} + \frac{11.888k(0.464ft(1.833ft))}{1.681(A)ft^2} \]

\[ P_T = 53.567 \text{ tons pile} \leq 56 \text{ tons pile} \quad \text{o.k.} \]

\[ P_H = \frac{11.888k}{0.250A} - \frac{11.888k(0.464ft(3.667ft))}{1.681(A)ft^2} = 35.519k \]

\[ P_H = 17.760 \text{ tons pile} \leq 56 \text{ tons pile} \quad \text{o.k.} \]

Check Sliding

\[ F.S._{Sliding} = \frac{3.285k+5.903k+0.832k}{0.902k+2.256k+0.714k} = 2.588 \geq 1.2 \quad \text{o.k.} \]

Case III

F.S. for overturning \( \geq 1.5 \)
F.S. for sliding \( \geq 1.5 \)

Check Overturning

\[ \sum M_R = (22.887 + 26.960 + 3.584)(ft-k) = 53.431(ft-k) \]

\[ \sum M_{OT} = 18.799(ft-k) \]

\[ F.S._{OT} = \frac{\sum M_R}{\sum M_{OT}} = \frac{53.431(ft-k)}{18.799(ft-k)} = 2.842 \geq 1.5 \quad \text{o.k.} \]

Check Pile Bearing

\[ e = \frac{\sum M}{\sum V} = \frac{(13.174+18.930-7.519)(ft-k)-(4.510+9.282)(ft-k)}{(5.828+4.820+1.240)k} = 2.309ft \]

Moment arm = 2.309ft - 1.833ft = 0.476ft

\[ P_T = \frac{10.648k}{0.250A} - \frac{10.648k(0.476ft(1.833ft))}{1.681(A)ft^2} = 37.065k \]

\[ P_T = 18.532 \text{ tons pile} \leq 56 \text{ tons pile} \quad \text{o.k.} \]

\[ P_H = \frac{10.648k}{0.250A} + \frac{10.648k(0.476ft(3.667ft))}{1.681(A)ft^2} = 53.649k \]

\[ P_H = 26.825 \text{ tons pile} \leq 56 \text{ tons pile} \quad \text{o.k.} \]

Check Sliding

\[ F.S._{Sliding} = \frac{3.285k+5.903k+0.832k}{2.256k} = 4.441 \geq 1.5 \quad \text{o.k.} \]
Case IV

Check Pile Bearing

\[ e = \frac{\sum M}{\sum V} = \frac{(13.174 + 18.930)(ft-k)}{5.628k \cdot 4.820k} = 3.015 \text{ft} \]

Moment arm = 3.015ft - 1.833ft = 1.182ft

\[ P_H = \frac{\sum V}{A} + \frac{Mc}{I} = \frac{10.648k}{0.250A} + \frac{10.648k \cdot 1.182ft \cdot 3.667ft}{1.681A/ft^2} = 70.047k = 35.024 \text{ tons pile} \]

PH = 70.047k = 35.024 tons pile ≤ 1.25(56 tons pile) = 70 tons pile o.k.

25% over-stress is allowed on the heel pile:

\[ P_H = 35.024 \text{ tons pile} \leq 1.25(56 \text{ tons pile}) = 70 \text{ tons pile} \quad \text{o.k.} \]

\[ P_T = \frac{10.648k}{1.681A/ft^2} = 10.648k \cdot 1.182ft \cdot 1.833ft \quad \text{28.868k} \quad \text{o.k.} \]

\[ P_H = 10.648k \cdot 1.2 \cdot (2.000 \text{ ft})^2 = 0.632k \text{ ft}^3 \]

Apply Load Factors

\[ F_{Col} = \gamma \beta \cdot LL (0.714k) = (1.3)(1.67)(0.714k) = 1.550k \]

\[ P_{LL} = \gamma \beta \cdot E (0.632k) = (1.3)(1.67)(0.632k) = 1.372k \]

\[ P_{A\text{Stem}} = \gamma \beta \cdot E (1.105k) = (1.3)(1.3)(1.105k) = 1.867k \]

\[ M_u = (10.00 \text{ ft})(1.550k) + (3.500 \text{ ft})(1.372k) + (2.333 \text{ ft})(1.867k) \]

Reinforcement - Stem

\[ b = 12 \text{ in} \]

\[ \text{cover} = 2 \text{ in} \]

\[ h = 16 \text{ in} \]

\[ d = 16 \text{ in} - 2 \text{ in} - 0.5(0.625 \text{ in}) = 13.688 \text{ in} \]

\[ F_{\text{Collision}} = 0.714 \frac{k}{\text{ft}} \]

\[ P_{LL} = \gamma sCAH(2.000 \text{ ft}) = (2.000 \text{ ft})(0.376)(7.000 \text{ ft})(0.120 \frac{k}{\text{ft}^3}) = 0.632 \frac{k}{\text{ft}} \]

\[ P_{A\text{Stem}} = \gamma sCAH(2) = \frac{1}{2} (0.120 \frac{k}{\text{ft}^3})(0.376)(7.000 \text{ ft})^2 = 1.105 \frac{k}{\text{ft}} \]
$M_u = 24.658(\text{ft} - k)$

$R_n = \frac{M_u}{\rho bd^2} = \frac{24.658(\text{ft} - k)}{(0.9)(1\text{ft})(13.688\text{in})^2} = 0.146\text{ksi}$

$\rho = \frac{0.85\sqrt{1 - \frac{1}{2}\frac{2R_n}{f_y}} f_y}{60\text{ksi}} = \frac{0.85(\sqrt{1 - \frac{2(0.146\text{ksi})}{0.85(3\text{ksi})}})}{60\text{ksi}} = 0.00251$

$R_{min} = 1.7\sqrt{\frac{h}{d}}^2 \frac{f_y}{f_y} = 1.7\left[\sqrt{\frac{16\text{in}}{13.688\text{in}}}\right]^2 \frac{3000\text{psi}}{60000\text{psi}} = 0.00212$

$\rho = 0.00251$

$A_{Sreq} = \rho bd = (0.00251)(12\text{in})(13.688\text{in}) = 0.412\text{in}^2$

One # 5 bar has $A_s = 0.307\text{in}^2$

$s = \frac{12\text{in}}{0.307\text{in}^2} = 4.12\text{in}^2$

$s = 8.9\text{in}$

Use # 5 bars @ 8.5 in. cts.

Check Shear

$V_u \leq \phi V_n$

$V_u = F_{\text{collision}} + P_{LL} + P_{A_{\text{stem}}} = 1.550k + 1.372k + 1.867k = 4.789k$

$\frac{V_u}{\phi bd} = \frac{V_u}{0.85(12\text{in})(13.688\text{in})} = 34.301\text{psi}$

$V_n = V_c = 2\sqrt{f_c} = 2\sqrt{3000\text{psi}} = 109.5\text{psi} > 34.3\text{psi} \quad \text{o.k.}$

Reinforcement - Footing - Top Steel

$b = 12\text{in}$

cover = 3 in

$h = 36\text{in}$

$d = 36\text{in} - 3\text{in} - 0.5(0.5\text{in}) = 32.750\text{in}$

Design the heel to support the entire weight of the superimposed materials.
Soil(1) = 4.340 \text{ k}\text{ft} \\
LL_s = 1.240 \text{ k}\text{ft} \\
Slab wt. = (3.000\text{ft})\left[0.150\frac{\text{ft}^2}{\text{ft}^2}\right](5.167\text{ft}) = 2.325 \text{ k}\text{ft}

Apply Load Factors

Soil(1) = \gamma \beta_E (4.340 \text{k}) = (1.3)(1.0)(4.340 \text{k}) = 5.642 \text{k} \\
LL_s = \gamma \beta_E (1.240 \text{k}) = (1.3)(1.67)(1.240 \text{k}) = 2.692 \text{k} \\
Slab wt. = \gamma \beta_D (2.325 \text{k}) = (1.3)(1.0)(2.325 \text{k}) = 3.023 \text{k}

\[ M_u = (2.583\text{ft})(5.642\text{k} + 2.692\text{k} + 3.023\text{k}) = 29.335(\text{ft} - \text{k}) \]

\[ R_n = \frac{M_u}{bd^2} = \frac{29.335(\text{ft} - \text{k})}{(0.9)(1\text{ft})(32.750\text{in})^2} = 0.0304\text{ksi} \]

\[ \rho = \frac{0.85(3\text{ksi})}{60\text{ksi}}\left[1 - \sqrt{\frac{1}{\frac{1}{60\text{ksi}} - \frac{20(0.0304\text{ksi})}{0.85(3\text{ksi})}}} \right] = 0.000510 \]

\[ \rho_{min} = 1.7\left(\frac{36\text{in}}{32.750\text{in}}\right)^2\left(\frac{3000\text{psi}}{60,000\text{psi}}\right) = 0.00188 \]

Use \( \rho = \frac{4}{3}\rho = \frac{4}{3}(0.000510) = 0.000680 \)

\[ A_{S_{req}} = \rho bd = (0.000680)(12\text{in})(32.750\text{in}) = 0.267\frac{\text{in}^2}{\text{ft}} \]

One # 4 bar has \( A_s = 0.196\text{in}^2 \)

\[ \frac{s}{0.196\text{in}^2} = \frac{12\text{in}}{0.267\text{in}^2} \]

\[ s = 8.8\text{in} \]

Use # 4 bars @ 8.5 in. cts.

Check Shear

\[ V_u = \text{Soil(1)} + LL_s + \text{Slab wt.} = 5.642\text{k} + 2.692\text{k} + 3.023\text{k} = 11.357\text{k} \]

\[ \frac{V_u}{\varphi} = \frac{V_u}{\varphi bd} = \frac{11357\text{lbs}}{0.85(12\text{in})(32.750\text{in})} = 33.998\text{psi} \leq 109.5\text{psi} = \nu_c \quad \text{o.k.} \]

Reinforcement - Footing - Bottom Steel

Design the flexural steel in the bottom of the footing to resist the largest moment that the heel pile could exert on the footing. The largest heel pile bearing force was in Case IV. The heel pile will cause a larger moment about the stem face than the toe pile (even though there are two toe piles for every one heel pile) because it has a much longer moment arm about the stem face.
Pile is embedded into footing 12 inches.
b = 12 in
h = 36 in
d = 36 in - 4 in = 32 in

Apply Load Factors to Case IV Loads

\[ \Sigma V = \gamma \beta \frac{5.828 k}{R_f} + \gamma \beta \frac{4.820 k}{R_f} \]
\[ \Sigma V = 1.3(1.0) \left( \frac{5.828 k}{R_f} \right) + 1.3(1.0) \left( \frac{4.820 k}{R_f} \right) \]
\[ \Sigma V = 13.842 \frac{k}{R_f} \]
\[ \Sigma M = \gamma \beta \frac{13.174 (h-k)}{R_f} + \gamma \beta \frac{18.930 (h-k)}{R_f} \]
\[ \Sigma M = (1.3)(1.0) \left( \frac{13.174 (h-k)}{R_f} \right) + (1.3)(1.0) \left( \frac{18.930 (h-k)}{R_f} \right) \]
\[ \Sigma M = 41.735 \frac{k}{R_f} \]

\[ e = \frac{\Sigma M}{\Sigma V} = \frac{41.735(h-k)}{13.842k} = 3.015\text{ft} \]

Moment arm = 3.015 ft - 1.833 ft = 1.182 ft

\[ P_H = \frac{\Sigma V}{A} + \frac{Mc}{I} = \frac{13.842k}{0.250A} + \frac{13.842k(1.182ft)(3.667ft)}{1.681(1/A)ft^2} \]
\[ P_H = 91.059 \frac{k}{\text{pt}} \left( \frac{1}{12\text{ft}} \right) = 7.588 \frac{k}{R_f} \]

\[ M_u = \left( \frac{7.588 k}{R_f} \right)(3.667ft) = 27.825 \frac{(h-k)}{R_f} \]

\[ R_n = \frac{M_u}{bd^2} = \frac{27.825(h-k)}{0.9(1/ft)(32in)^2} = 0.0301ksi \]

\[ \rho = \frac{0.85(3ksi)}{60ksi} \left[ 1 - \sqrt{1 - \left( \frac{210 \times 0.0301 \text{ksi}}{0.85(3ksi)} \right) } \right] = 0.000505 \]

\[ \rho_{\text{min}} = 1.7 \left( \frac{36in}{32in} \right)^2 \frac{3,000 \text{psi}}{60,000 \text{psi}} = 0.00196 \]

Use \( \rho = \frac{4}{3} \rho = \frac{4}{3}(0.000505) = 0.000673 \)

\[ A_{S_{\text{req}}} = \rho bd = (0.000673)(12\text{in})(32\text{in}) = 0.258 \frac{in^2}{R_f} \]
One # 4 bar has $A_s = 0.196 \text{ in}^2$.

\[
s = \frac{12 \text{ in}}{0.196 \text{ in}^2} = 61.22 \text{ in}
\]

Use # 4 bars @ 9 in. cts.

**Check Shear**

The critical section for shear for the toe is at a distance $d = 21.75$ inches from the face of the stem. The toe pile is 6 inches from the stem face so the toe pile shear does not affect the shear at the critical section. The critical section for shear is at the stem face for the heel so all of the force of the heel pile affects the shear at the critical section. The worst case for shear is Case IV.

\[
V_u = 7.588k
\]

\[
\frac{V_u}{V_{0}} = \frac{V_u}{0.85(12\text{in})(32\text{in})} = 23.248 \text{ psi} \leq 109.5 \text{ psi} = v_c \quad \text{o.k.}
\]

**Reinforcement - Shear Key**

![Diagram of reinforcement shear key]

- $b = 12 \text{ in}$
- $h = 12 \text{ in}$
- cover = 3 in
- $d = 12 \text{ in} - 3 \text{ in} - 0.5(0.5 \text{ in}) = 8.75 \text{ in}$

**Apply Load Factors**

\[
P_P = \gamma \beta E(3.845k) = (1.3)(1.3)(3.845k) = 6.498k
\]

\[
M_u = (0.912\text{ft})(6.498k) = 5.926(\text{ft} - k)
\]

\[
R_n = \frac{M_u}{0.9d^2} = \frac{5.926(\text{ft} - k)}{(0.9)(1\text{ft})(8.75\text{in})^2} = 0.0860 \text{ ksi}
\]

\[
\rho = \frac{0.85(3\text{ksi})}{60\text{ksi}} \left[ 1 - \sqrt{1 - \frac{2(0.0860\text{ksi})}{0.85(3\text{ksi})}} \right] = 0.00146
\]

\[
\rho_{\min} = 1.7 \left[ \frac{12\text{in}}{8.75\text{in}} \right]^2 \frac{\sqrt{3,000\text{psi}}}{60,000\text{psi}} = 0.00292
\]
Use $\rho = \frac{4}{3} \rho = \frac{4}{3}(0.00146) = 0.00195$

$$A_{S\text{req}} = \rho bd = (0.00195)(12\text{in})(8.75\text{in}) = 0.205\text{in}^2$$

One # 4 bar has $A_s = 0.196\text{in}^2$

$$\frac{s}{0.196\text{in}^2} = \frac{12\text{in}}{0.205\text{in}^2}$$

$s = 11.5\text{in}$

Use # 4 bars @ 11 in. cts.

**Check Shear**

$$\nu_u \varphi = \frac{V_u}{\varphi bd} = \frac{6498\text{lbs}}{0.85(12\text{in})(8.75\text{in})} = 72.807\text{psi} < 109.5\text{psi} = \nu_c$$

**Reinforcement Summary**

![Diagram of Reinforcement Summary]
Bridge Manual

Retaining Walls - Section 3.62

3.6 Dimensions  Cast-In-Place Concrete Retaining Walls

Cont'd

Each section of wall shall be in increments of 4 feet with a maximum length of 28'-0".

Slope as specified on Design Layout.

Vertical or battered (As req'd, by design)

As required by design

(Use 1/2 "H" as estimate)

TYPICAL SECTION THRU WALL (SPREAD FOOTING)

(*) 10'-1/2" is minimum for median walls.

(**) 15" is minimum for median walls.

For SPC B, C, and D follow guidelines for footing depth and pile embedment as given in Sec. 3.71 Page 5.1-4. See also Sec. 3.71 Page 5.3.3 and Sec. 3.74 Page 1.2.4 for channel shear connectors and anchorage reinforcement.

TYPICAL PART SECTION THRU WALL (PILE FOOTING)

(friction or bearing piles)

For Piles, 18" or greater, as required.

Revised: Dec. 1998
Each section of wall shall be in increments of 4 feet with a maximum length of 28'-0".

10" (Min.) (*)

Slope as specified on Design Layout.

Vertical or battered (As req'd. by design)

"L" as required by design

(Use $\frac{1}{2}"$ as estimate)

TYPICAL SECTION THRU WALL
(Spread footing)

(*) 10-1/2" is minimum for median walls.
(**) 15" is minimum for median walls.
(*** Due to site constriction)

For SPC A, C and D follow guidelines for footing depth and pile embedment as given in Sec. 3.71 Page 5.1.4. See also Sec. 3.71 Page 5.3.3 and Sec. 3.74 Page 1.2.4 for channel shear connectors and anchorage reinforcement.

TYPICAL PART SECTION THRU WALL
(Pile footing)

(Concrete pedestal pile)

(Vertical friction or bearing pile) (***)

Revised: Dec. 1998

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Each section of wall shall be in increments of 4 feet with a maximum length of 28' 0".

TYPICAL SECTION THRU WALL
(Spread footing)
Counterfort Walls

"A"

"B"  "C"  "C"  "B"

15" (Min.)

Continuous footing

PART ELEVATION

Note: For "A", "B", "C", "D", and "L" see Page: 3.6-6.
Sign-Board Type Counterfort Walls

"A" (As required by Design Layout)

"B"

"C"

"D"

18" (Min.)

"E"

PART ELEVATION

Note: For "A", "B", "C", "D", "E", and "L" see the following sheet.

Slope as specified on Design Layout.

9" (Min.)

Batter "D" foot

12" (Min.)

Finished ground line

12" (Min.)

18" (Min.)

2'-0" (Min.)

TYPICAL SECTION THRU WALL & SPREAD FOOTING

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Dimension "A"

Maximum length = 28'-0".
Each section to be in 4'-0" increments.
(See rustication recess details, Sec. 3.62 Page 3.8-3)

Dimensions "B" & "C"

As required by the design to balance the negative and positive moments. (See the design assumptions).

Batter "D"

As required to maintain 9" minimum at top of counterfort and 12" minimum edge distance at the top of the footing, between counterfort and footing edge.
Batter to be given in eighth of an inch per foot of counterfort height.

Dimension "E"

(Sign-board type only)
As required to maintain footing pressure within the allowable for existing foundation material. 12" minimum.

Dimension "L"

As required for stability.
As an estimate, use "L" equal to \( \frac{1}{2} \) the height of "H".
3.7 Reinforcement

Cast-In-Place Concrete Retaining Walls

Cantilever Walls

3" C.i. (Typ.)

By design (\#4 @ 12" Min.)

Typical Section Thru Wall
(Spread Footing)

\#4 @ 12" cts.

By design
(\#4 @ 12" Min.)

Typical Part Section Thru Wall
(Pile Footing)
(Friction or bearing piles)

(\#) Alternate long and short bars at equal spaces.
(\#\#) If collision forces are assumed, use \#4 @ 12" cts. Min., and extend at least development length into footing. (See Section 2.4).
(\#\#\#) Theo. cut-off for bending + development length (Wall height over 10' only).

Revised: Dec. 1998
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Cantilever Walls - L-Shaped

Cost-In-Place Concrete Retaining Walls

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(※) Do not splice stress bars in the fill face at top of footing.

(※※) If collision forces are assumed, use #4 @ 12" cts., and extend at least development length into footing. (See Section 2.4).

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**Retaining Walls - Section 3.62**

**Counterfort Walls**

**Wall and Stem:**

By design, each face: 

- @ 18” cts. (Each face)
- 2” Cl.
- 1 ½” Cl.

By design:

- Stirrups (In pairs) by design
- @ 12” cts. (Min.)

Dowel bars (Same size and spacing as vertical wall steel)

**TYPICAL SECTION NEAR COUNTERFORT**

(For footing reinforcement, see the following sheet)

**PART PLAN SECTION A-A**

(*: Use development length (other than top bar) or standard hook with (Min.) tension emeement “E” (See Manual Section 2.41).

(**): See lap splices Class B (Other than top bars) (See Manual Section 2.41).
Footing:

(*) By design for loads and footing pressures on section under consideration. (M5 @ 12” cts. is the minimum.)

Revised: Dec. 1998
Counterfort Walls - Sign-Board Type

Wall and Stem:
Refer to preceding page.

Spread Footing:

![Diagram showing front and side views of a spread footing with counterforts and shear lines.]

If the shear line is within the counterfort projected (longitudinally or transversely) the footing may be considered satisfactory for all conditions. If outside of the counterfort projected, the footing must be analyzed and reinforced for bending and checked for bond stress and for diagonal tension stress.

TYPICAL SECTION THRU FOOTING

Revised: Dec. 1998
3.8 Details

Cast-In-Place Concrete Retaining Walls

Non-Keyed Joints

Each section of wall shall be in increments of 4 feet with a maximum length of 28'-0".

Expansion and contraction joints:

| Between exp. joints (wall) | 28' (Maximum) | 4' Joint |

PART ELEVATION

(*) Between expansion joint and contraction joint or between contraction joint and contraction joint.

(**) Footings are to be constructed continuous if below frost line. Footings at the surface are to match expansion joints in wall.

Fill back of 1/2" joint with joint sealer

1/2" Preformed fiber exp. joint material

4" Plastic waterstop

3/4" Chamfer

Horizontal reinforcement (Typ.)

1" (Typ.)

PLAN SECTION THRU WALL

See Manual Section 4.0 for appropriate notes.
**Bridge Manual**

**Retaining Walls - Section 3.62**

**Cast-In-Place Concrete Retaining Walls**

### Keyed Joints

**Expansion and contraction joints:**

- Expansion joint
- Contraction joint

#### PART ELEVATION

- 8' (Maximum) wall height limit for keyed expansion joint or contraction joint. Use keyed joints if wall heights allow.

- Fill back into 1/2" joint with joint sealer

- 1/2" Preformed fiber exp. joint material

- 4" Plastic waterstop

- Front face 1/2" Chamfer

#### SECTION A-A

**Expansion joint**

- Horizontal reinforcement (Typ.)

#### SECTION B-B

**Contraction joint**

- Equal spaces

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**Effective:** Feb. 2, 2004  **Supersedes:** Dec. 1998  **E3.62-0**
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Cast-In-Place Concrete Retaining Walls

Rustication Recess

3/4" bevel

Rustication recess at 4°0' cts.

Note: Use rustication in urban areas.

PART ELEVATION

SECTION A-A

SECTION B-B

SECTION C-C

SECTION D-D

FILLED JOINT DETAIL
(For wall where rustication is not required)

(*) 1/2" Preformed fiber expansion joint material shall be in accordance with Standard Specifications 1057.


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Cast-In-Place Concrete Retaining Walls

Drains

3" Ø weephole (slope weephole ½") per ft. to drain

Finished ground line

FRENCH DRAIN

(* Limits of demarcating (Use only when specified on the Design Layout.)

Front face

6" Rustication recess 3" Ø weephole @ abt. 8-0" cts. (Min.)

Finished ground line

WEEPHOLE DETAIL

4", 6" or 8" (from Design Layout) perforated corrugated metal pipe (60° v.) underdrain – full length of wall

PCMP UNDERDRAIN

Note: French drains shall be used on all retaining walls, unless otherwise specified on the Design Layout.

When drop inlets go through the footings of retaining walls, use the following note:

Note: Contractor may shift and or cut reinforcement in field where necessary to clear drains. Place 1/2" joint filler around drop inlet as bond breaker.
Construction Joint Keys

Cantilever Walls:

Continuous conn. joint
(Key 2" X 4", nominal)

Counterfort Walls:

Continuous conn. joint
(Key 2" X 4", nominal)

Key length: Divide the length A into an odd number of spaces of equal lengths. Each space shall not exceed 24 inches in length. Use as few spaces as possible with the minimum number of spaces equal to three (or one (1) key).

Key width = Counterfort width/3 (to the nearest inch)

Key depth = 2" (Nominal)
**Bridge Manual**

**Retaining Walls - Section 3.62**

*Cast-In-Place Concrete Retaining Walls*

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**Sign-Board Walls:**

- Keyed const. joint

**SECTION A-A**

- Key width = "C"/3 (To the nearest inch)
- Key depth = 2" (Nominal)

- Equal spaces
- Key length

- Keyed const. joint

- Equal spaces
- Key length

**Key length = divide length "A" or "B" into an odd number of spaces of equal lengths. Each space shall not exceed 24" in length. Use as few spaces as possible with the minimum number of spaces equal to three (or one [1] key).**

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**Revised: Dec. 1998**