DESIGN OF HIGHWAY BRIDGE ABUTMENTS AND FOUNDATIONS

PART 2

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DISCLAIMER

ACKNOWLEDGEMENTS

This project is funded by the Michigan Department of Transportation. This project is carried out by the Michigan Department of Transportation Center of Excellence for Structural Durability at Western Michigan University. We, the authors would like to acknowledge Mr. Juan Alcantar as the project manager and the advisory panel members for their assistance in finding relevant information, reviewing the drafts, and responding to our questions in a timely manner to successfully complete this example. We appreciate the contribution of Ms. Kanchani Basnayake and Mr. Harsha Amunugama to the development of this example.

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Section 3 Abutment with Spread Footing and EPS Backfill

Step 3.1 Preliminary Abutment Dimensions

Description

This step presents the selected preliminary abutment dimensions.

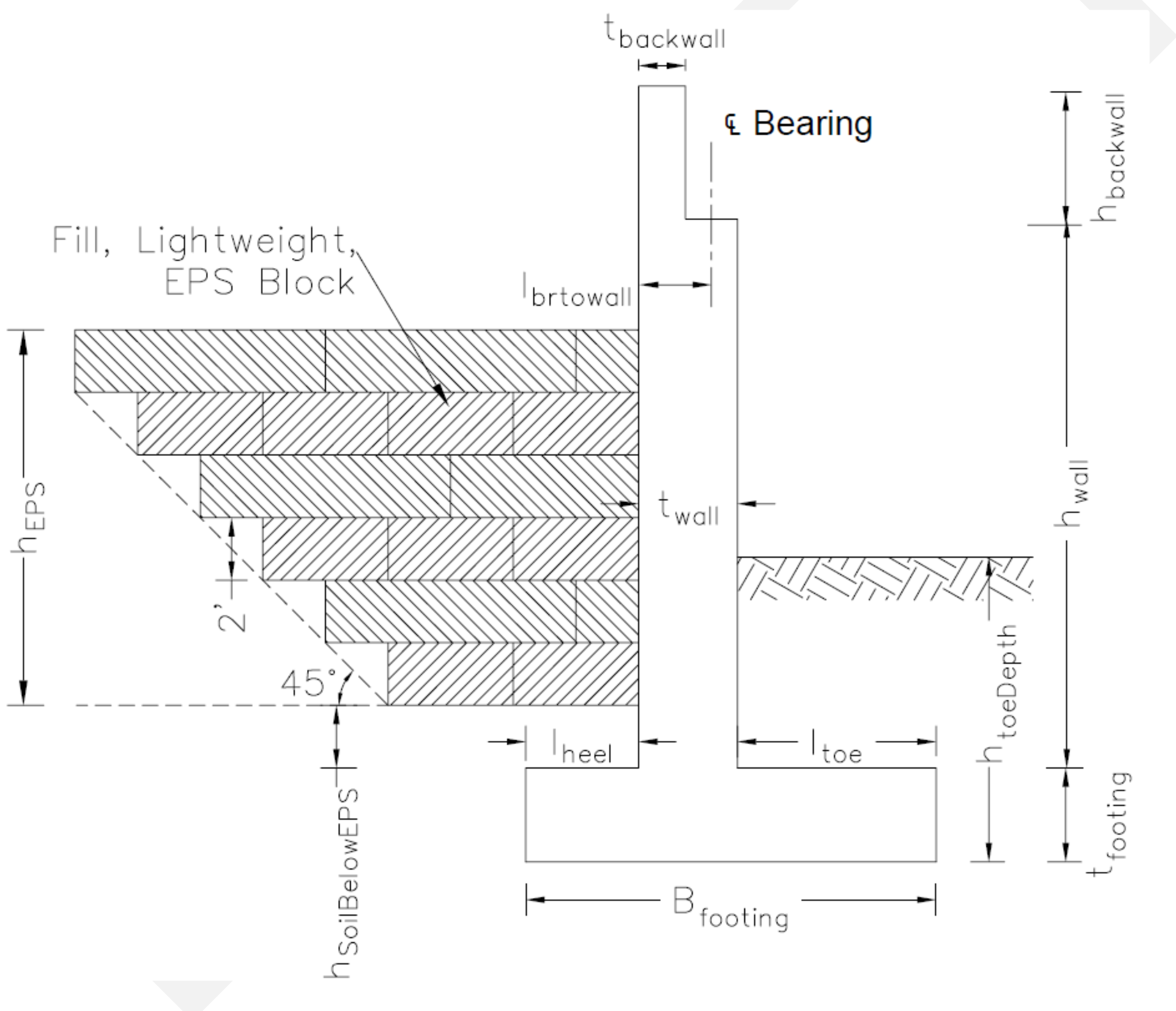
The design criteria, bridge information, material properties, reinforcing steel cover requirements, soil types and properties, and superstructure loads in this section are taken from Section 2.

This section presents the design of a full-depth reinforced concrete cantilever abutment with expanded polystyrene (EPS) blocks as the lightweight backfill material.

Geofoam made with EPS is effective at reducing lateral forces or settlement potential for bridge abutments (MDOT Geotechnical Manual 2019). The selection of a specific Geofoam grade depends on the project needs. A typical Geofoam embankment consists of the foundation soils, the Geofoam fill, and a load dissipater slab designed to transfer loads to the Geofoam.

Design guidelines for Geofoam embankments are provided in the National Cooperative Highway Research Program (NCHRP) web document 65, titled *Geofoam Applications in the Design and Construction of Highway Embankments (Stark et al., 2004).* It is cited as **NCHRP w65** in this design example.

The designer should select the preliminary dimensions based on state-specific standards and past experience. The following figure shows the abutment geometry and dimensional variables:



The selected preliminary dimensions are listed below.

Abutment length $L_{abut} := W_{deck} = 63.75\ ft$

This abutment has an independent backwall with a sliding deck slab. **BDG 6.20.03A**

Backwall height $h_{backwall} := 4.25ft$

Backwall thickness $t_{backwall} := 1ft + 6in = 1.5\ ft$

Abutment wall height $h_{wall} := 17.54ft$

Abutment wall thickness $t_{wall} := 3ft + 2in = 3.167\ ft$

Distance from the toe to the front face of the abutment wall $l_{toe} := 6ft + 4in = 6.333\ ft$

Distance from the heel to the back face of the abutment wall $l_{heel} := 4ft$

Distance from center of the bearing pad to the back face of the abutment wall $l_{brtowall} := 2ft + 4in = 2.333\ ft$

Footing width $B_{footing} := l_{toe} + l_{heel} + t_{wall} = 13.5\ ft$

Footing length $L_{footing} := L_{abut} + 1ft + 1ft = 65.75\ ft$

Note: The footing is extended 1 ft beyond the end of the wall on either side.

Footing thickness $t_{footing} := 3ft$

Toe fill depth to the bottom of the footing $h_{toeDepth} := 7ft$

Note: Bottoms of footings are normally set 4 ft below the existing or proposed ground lineto avoid frost heave. **BDM 7.03.02 D**

. Passive earth pressure is excluded from the footing design. **BDM 7.03.02 F**

For the backfill EPS blocks, assume the following properties:

Unit weight of EPS blocks $\gamma_{EPS} := 2\frac{lb}{ft^3}$ **Michigan Geotechnical Manual page 109**

Slope angle of the EPS block end profile $\theta := 45deg$

Internal friction angle of the backfill soil $\phi := 32deg$

The friction angle of EPS/soil interface is typically assumed to be equal to ϕ. **NCHRP w65**

Friction angle of EPS/soil interface $\delta := \phi = 32\cdot deg$

Active lateral earth pressure coefficient for the EPS blocks (based on the Coulomb's classical earth-pressure theory)

$$k_{EPS} := \left(\frac{\sin(\theta - \phi) \cdot \frac{1}{\sin(\theta)}}{\sqrt{\sin(\theta + \delta)} + \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi)}{\sin(\theta)}}} \right)^2$$

$k_{EPS} = 0.031$ **NCHRP w65**

Height of the EPS blocks $h_{EPS} := 12\text{ft}$

Depth of the soil between bottom of the EPS blocks and top of the footing $h_{SoilBelowEPS} := 2\text{ft}$

Height of backfill soil above the EPS blocks $h_{SoilAboveEPS} := h_{wall} + h_{backwall} - h_{EPS} - h_{SoilBelowEPS} = 7.79\text{ ft}$

If the fill above the EPS blocks is greater than 8 ft, the compressive strength of the blocks needs to be checked.

According to the MDOT Geotechnical Manual (2019), EPS blocks should not be used where the water table could rise and make geoform unstable due to buoyant forces. The structural engineer and the geotechnical engineer need to work together and check EPS stability for a 100-year flood.

Step 3.2 Application of Dead Load

Description

This step describes the application of the dead load on the abutment.

The common practice is to apply superstructure dead load as a uniformly distributed load over the length of the abutment. This is accomplished by adding exterior and interior girder end dead load reactions and dividing this quantity by the abutment length.

Dead load of superstructure

Weight of structural components and non-structural attachments (DC)

$$DC_{Sup} := \frac{2 \cdot R_{DCEx} + \left(N_{beams} - 2\right) \cdot R_{DCIn}}{L_{abut}} = 5.658 \cdot \frac{kip}{ft}$$

Weight of the future wearing surface (DW)

$$DW_{Sup} := \frac{2 \cdot R_{DWEx} + \left(N_{beams} - 2\right) \cdot R_{DWIn}}{L_{abut}} = 0.886 \cdot \frac{kip}{ft}$$

Backwall weight

$$DC_{backwall} := h_{backwall} \cdot t_{backwall} \cdot W_c = 0.956 \cdot \frac{kip}{ft}$$

Abutment wall weight

$$DC_{wall} := h_{wall} \cdot t_{wall} \cdot W_c = 8.332 \cdot \frac{kip}{ft}$$

Footing weight

$$DC_{footing} := B_{footing} \cdot t_{footing} \cdot W_c = 6.075 \cdot \frac{kip}{ft}$$

Step 3.3 Application of Live Load

Description

Please refer to Step 2.3. The same loads are applied following the procedures described in Step 2.3.

Step 3.4 Application of Other Loads

Description

This step typically includes the calculation of braking force, wind load, earth load, and temperature load.

The calculation of 'other loads', except the earth load, is identical to Step 2.4. Since EPS blocks are used as the backfill and a different spread footing width is selected, the calculation of the earth load is different. Therefore, this step only presents the earth load calculation. Please refer to Step 2.4 for the rest of the calculations.

Earth Load

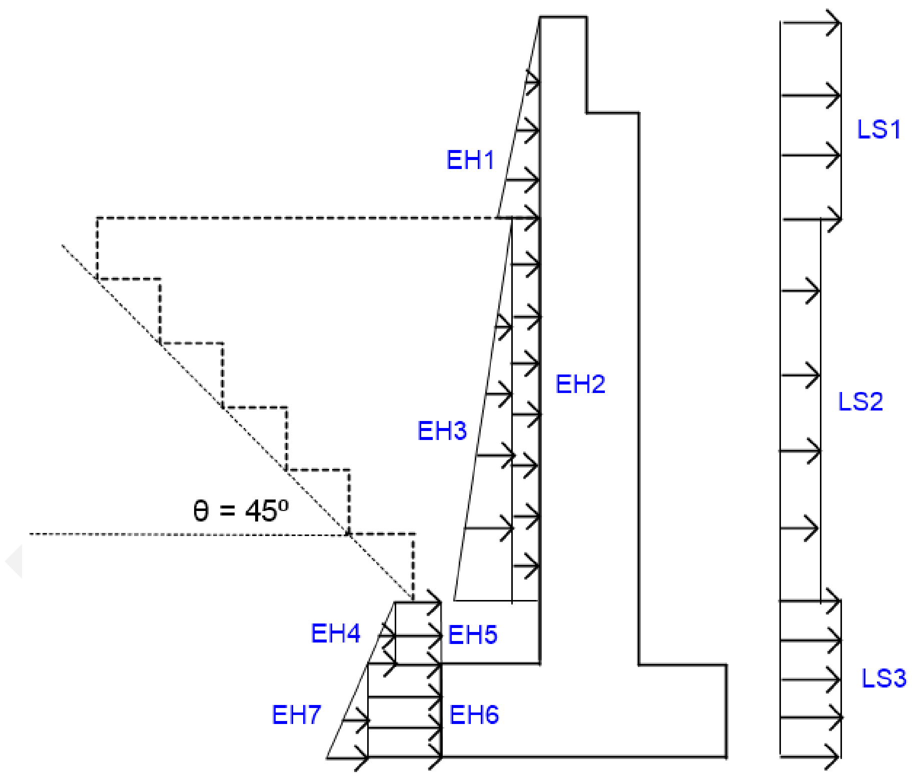
The earth load includes lateral earth pressure, live load surcharge, and vertical earth pressure on the footing. As per the geotechnical engineer, the groundwater table is not located in the vicinity of the foundation. Therefore, the effect of hydrostatic pressure is excluded. If possible, the hydrostatic pressure should be avoided at abutments and retaining walls through the design of an appropriate drainage system.

Lateral Load Due to Lateral Earth Pressure

The lateral pressure and the resultant force due to earth pressure are calculated.

The lateral component of the earth load on the abutment consists of seven parts as listed below and shown in the following figure:

EH 1: the lateral pressure from the soil above the EPS blocks
EH 2: the lateral pressure due to the vertical load at the top of the EPS blocks
EH 3: the lateral pressure from the soil behind the EPS blocks
EH 4: the lateral pressure from the soil located below the EPS blocks and above the top of the footing
EH 5: the lateral pressure due to the vertical load at the bottom of the EPS blocks
EH 6: the lateral pressure due to the vertical load at the top of the footing
EH 7: the lateral pressure from the soil located along the depth (thickness) of the footing.



Backwall

Lateral earth pressure at the base $p_{bw} := k_a \cdot \gamma_s \cdot h_{backwall} = 0.153 \cdot ksf$ **LRFD Eq. 3.11.5.1-1**

Lateral load $P_{EHBackwall} := \frac{1}{2} \cdot p_{bw} \cdot h_{backwall} = 0.325 \cdot \frac{kip}{ft}$

Abutment Wall

The calculation of lateral loads on the abutment wall with EPS blocks as the backfill follows the procedure outlined in the NCHRP web document 65, titled *Geoform Applications in the Design and Construction of Highway Embankments* by Stark et al. (2004).

Height of backfill soil above the EPS blocks $h_{SoilAboveEPS} = 7.79\ ft$

EH 1: **LRFD Eq. 3.11.5.1-1**

Lateral earth pressure at the top of EPS blocks $p_{AboveEPS} := k_a \cdot \gamma_s \cdot h_{SoilAboveEPS} = 0.28 \cdot ksf$

Lateral load from the soil above the EPS blocks $P_{EH1} := \frac{1}{2} \cdot p_{AboveEPS} \cdot h_{SoilAboveEPS} = 1.092 \cdot \frac{kip}{ft}$

EH 2:

Lateral earth pressure due to the vertical load at the top of the EPS blocks $p_{VEPS} := \frac{1}{10} \gamma_s \cdot h_{SoilAboveEPS} = 0.093 \cdot ksf$ **NCHRP w65**

Lateral load due to the vertical load at the top of the EPS blocks $P_{EH2} := p_{VEPS} \cdot h_{EPS} = 1.122 \cdot \frac{kip}{ft}$

EH 3:

Lateral earth pressure of the soil behind the EPS blocks $p_{SoilBehind} := k_{EPS} \gamma_s \cdot h_{EPS} = 0.045 \cdot ksf$

Lateral load from the soil behind the EPS blocks $P_{EH3} := \frac{1}{2} \cdot p_{SoilBehind} \cdot h_{EPS} = 0.268 \cdot \frac{kip}{ft}$

EH 4:

Lateral earth pressure of the soil locate below the EPS blocks and over the top of the footing $p_{SoilBelowEPS} := k_a \cdot \gamma_s \cdot h_{SoilBelowEPS} = 0.072 \cdot ksf$

Lateral load from the soil locate below the EPS blocks and over the top of the footing $P_{EH4} := \frac{1}{2} \cdot p_{SoilBelowEPS} \cdot h_{SoilBelowEPS} = 0.072 \cdot \frac{kip}{ft}$

EH 5:

Lateral earth pressure due to the vertical load at the bottom of the EPS blocks $p_{VSoilBelowEPS} := k_a \cdot \left(\gamma_s \cdot h_{SoilAboveEPS} + \gamma_{EPS} \cdot h_{EPS}\right) = 0.288 \cdot ksf$

Lateral load due to the vertical load at the bottom of the EPS blocks $P_{EH5} := p_{VSoilBelowEPS} \cdot h_{SoilBelowEPS} = 0.575 \cdot \frac{kip}{ft}$

Total resultant lateral load at the base of the wall $P_{EHWall} := P_{EH1} + P_{EH2} + P_{EH3} + P_{EH4} + P_{EH5} = 3.129 \cdot \frac{kip}{ft}$

Total moment of the lateral earth load at the base of the wall

$$M_{EHWall} := P_{EH1} \cdot \left(\frac{1}{3} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS}\right) + P_{EH2} \cdot \left(\frac{1}{2} \cdot h_{EPS} + h_{SoilBelowEPS}\right) \cdots$$
$$+ P_{EH3} \cdot \left(\frac{1}{3} \cdot h_{EPS} + h_{SoilBelowEPS}\right) + P_{EH4} \cdot \frac{1}{3} \cdot h_{SoilBelowEPS} \cdots$$
$$+ P_{EH5} \cdot \frac{1}{2} \cdot h_{SoilBelowEPS}$$

$$M_{EHWall} = 29.331 \cdot \frac{kip \cdot ft}{ft}$$

<u>Footing</u>

The lateral earth load on the backwall, abutment wall, and footing are defined using 7 profiles. The forces acting on the abutment wall from profiles 1 to 5 remain unchanged. Hence, the calculation of forces from the earth load profiles 6 and 7 is described below.

EH 6:

Lateral earth pressure due to the vertical load at the footing top surface elevation

$$p_{SoilAboveFt} := k_a \cdot \left(\gamma_s \cdot h_{SoilAboveEPS} \cdots + \gamma_{EPS} \cdot h_{EPS} + \gamma_s \cdot h_{SoilBelowEPS}\right) = 0.36 \cdot ksf$$

Lateral earth load due to the vertical load at the footing top surface elevation

$$P_{EH6} := p_{SoilAboveFt} \cdot \left(t_{footing}\right) = 1.079 \cdot \frac{kip}{ft}$$

EH 7:

Lateral earth pressure due to the soil on the side of the footing

$$p_{SoilSideFt} := k_a \cdot \left(\gamma_s \cdot t_{footing}\right) = 0.108 \cdot ksf$$

Lateral earth load due to the soil on the side of the footing

$$P_{EH7} := \frac{1}{2} \cdot p_{SoilSideFt} \cdot t_{footing} = 0.162 \cdot \frac{kip}{ft}$$

Total lateral earth load

$$P_{EHFooting} := P_{EHWall} + P_{EH6} + P_{EH7} = 4.37 \cdot \frac{kip}{ft}$$

Total moment of the lateral earth load at the base of the footing

$$M_{EHFooting} := P_{EH1} \cdot \left(\frac{1}{3} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS} + t_{footing}\right) \cdots$$
$$+ P_{EH2} \cdot \left(\frac{1}{2} \cdot h_{EPS} + h_{SoilBelowEPS} + t_{footing}\right) + P_{EH3} \cdot \left(\frac{1}{3} \cdot h_{EPS} + h_{SoilBelowEPS} + t_{footing}\right) \cdots$$
$$+ P_{EH4} \cdot \left(\frac{1}{3} \cdot h_{SoilBelowEPS} + t_{footing}\right) + P_{EH5} \cdot \left(\frac{1}{2} \cdot h_{SoilBelowEPS} + t_{footing}\right) \cdots$$
$$+ P_{EH6} \cdot \frac{1}{2} \cdot \left(t_{footing}\right) + P_{EH7} \cdot \frac{1}{3} \cdot \left(t_{footing}\right)$$

$$M_{EHFooting} = 40.499 \cdot \frac{kip \cdot ft}{ft}$$

Vertical Earth Load on the Footing

Back side (heel)

$$EV_{earthBk} := \gamma_s \cdot l_{heel} \cdot \left(h_{SoilAboveEPS} + h_{SoilBelowEPS}\right) + \gamma_{EPS} \cdot l_{heel} \cdot h_{EPS}$$

$$EV_{earthBk} = 4.795 \cdot \frac{kip}{ft}$$

Front side (toe) $EV_{earthFt} := \gamma_s \cdot l_{toe} \cdot (h_{toeDepth} - t_{footing}) = 3.04 \cdot \frac{kip}{ft}$

Live Load Surcharge

A surcharge is applied to account for a vehicular live load acting on the backfill surface within a distance equal to one-half the wall height behind the back face of the wall. **LRFD 3.11.6.4**

The lateral component of the live load surcharge on the abutment wall consists of three parts, as shown in the previous figure.

LS 1: the lateral pressure from the soil above the EPS blocks
LS 2: the lateral pressure across the EPS block due to soil above it
LS 3: the lateral pressure from the soil below the EPS block

Height of the abutment $h_{backwall} + h_{wall} + t_{footing} = 24.79\ ft$

Equivalent height of soil for vehicular load $h_{eq} := 2ft$ **LRFD Table 3.11.6.4-1**

Lateral surcharge pressure $\sigma_p := k_a \cdot \gamma_s \cdot h_{eq} = 0.072 \cdot ksf$ **LRFD Eq. 3.11.6.4-1**

Backwall

Lateral load $P_{LSBackwall} := \sigma_p \cdot h_{backwall} = 0.306 \cdot \frac{kip}{ft}$

Abutment wall

Lateral load from the profile LS1 $P_{LSWall1} := \sigma_p \cdot h_{SoilAboveEPS} = 0.561 \cdot \frac{kip}{ft}$

Lateral load from the profile LS2 $P_{LSWall2} := \frac{1}{10} \gamma_s \cdot h_{eq} \cdot h_{EPS} = 0.288 \cdot \frac{kip}{ft}$ **NCHRP w65**

Lateral load from the profile LS3 $P_{LSWall3} := \sigma_p \cdot h_{SoilBelowEPS} = 0.144 \cdot \frac{kip}{ft}$

Total lateral load due to live load surcharge $P_{LSWall} := P_{LSWall1} + P_{LSWall2} + P_{LSWall3} = 0.993 \cdot \frac{kip}{ft}$

Total moment at the base of the wall due to the lateral component of the live load surcharge

$$M_{LSWall} := P_{LSWall1} \cdot \left(\frac{1}{2} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS}\right) \cdots$$
$$+ P_{LSWall2} \cdot \left(\frac{1}{2} \cdot h_{EPS} + h_{SoilBelowEPS}\right) + P_{LSWall3} \cdot \left(\frac{1}{2} h_{SoilBelowEPS}\right)$$

$$M_{LSWall} = 12.485 \cdot \frac{kip \cdot ft}{ft}$$

Footing

The lateral component of the live load surcharge on the footing consists of three parts. The contribution of LS 1 and LS 2 are the same as the abutment wall. The contribution of LS 3 needs to be considered up to the bottom of the footing.

Lateral surcharge load from the profile LS 1

$$P_{LSFooting1} := \sigma_p \cdot h_{SoilAboveEPS} = 0.561 \cdot \frac{kip}{ft}$$

Lateral surcharge load from the profile LS 2

$$P_{LSFooting2} := \frac{1}{10} \gamma_s \cdot h_{eq} \cdot h_{EPS} = 0.288 \cdot \frac{kip}{ft}$$

Lateral surcharge load from the profile LS 3

$$P_{LSFooting3} := \sigma_p \cdot \left(h_{SoilBelowEPS} + t_{footing}\right) = 0.36 \cdot \frac{kip}{ft}$$

Total lateral load due to live load surcharge

$$P_{LSFooting} := P_{LSFooting1} + P_{LSFooting2} + P_{LSFooting3} = 1.209 \cdot \frac{kip}{ft}$$

Total moment at the base of the footing due to the lateral component of the live load surcharge

$$\begin{aligned} M_{LSFooting} := {} & P_{LSFooting1} \cdot \left(\frac{1}{2} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS} + t_{footing}\right) \cdots \\ & + P_{LSFooting2} \cdot \left(\frac{1}{2} \cdot h_{EPS} + h_{SoilBelowEPS} + t_{footing}\right) \cdots \\ & + P_{LSFooting3} \cdot \frac{1}{2} \cdot \left(h_{SoilBelowEPS} + t_{footing}\right) \end{aligned}$$

$$M_{LSFooting} = 15.788 \cdot \frac{kip \cdot ft}{ft}$$

Vertical load

$$V_{LSFooting} := \gamma_s \cdot l_{heel} \cdot h_{eq} = 0.96 \cdot \frac{kip}{ft}$$

Step 3.5 Combined Load Effects

Description

This step presents the procedure for combining all load effects and calculating total factored forces and moments acting at the base of the abutment wall and footing. The total factored forces and moments at the base of the backwall are similar to the ones in Step 2.5.

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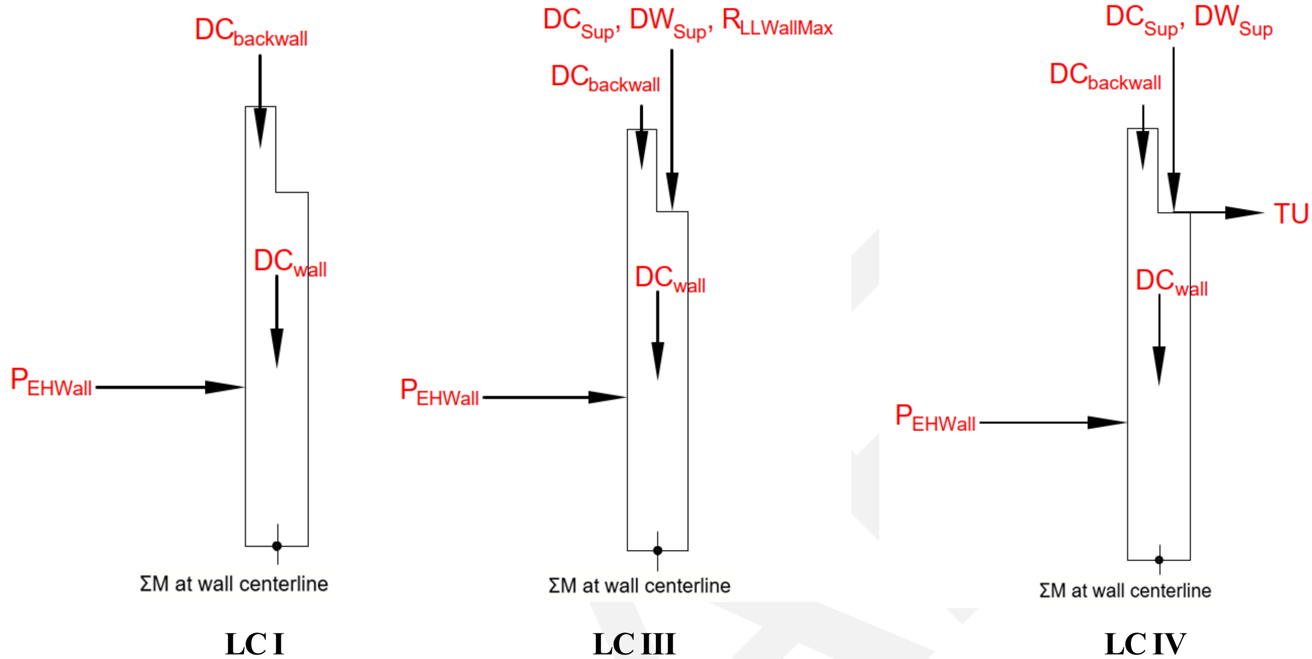
Forces and Moments at the Base of the Abutment Wall

Load Cases I, III, and IV are considered. Superstructure dead load, superstructure live load, and uniform temperature induced loads are considered in addition to the dead load of the backwall, dead load of the abutment wall, lateral earth pressure, and lateral surcharge pressure.



Strength I

Strength I = 1.25DC + 1.5DW + 1.75LL + 1.75BR + 1.5EH + 1.35EV + 1.75 LS + 0.5TU

Load Case I

Factored vertical force at the base of the wall

$$F_{VWallLC1StrI} := 1.25\cdot\left(DC_{backwall} + DC_{wall}\right) = 11.61\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC1StrI} := 1.5\cdot P_{EHWall} = 4.693\cdot\frac{kip}{ft}$$

The backwall weight reduces the critical moment at the base of the abutment wall. This requires the use of the minimum load factor of 0.9 for DC instead of the factor 1.25 in the Strength I combination.

LRFD 3.4.1

This is the same for the moment calculated about the longitudinal axis of the abutment wall for all the load cases and limit states.

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC1StrI} := 0.9\cdot DC_{backwall}\cdot\frac{\left(t_{backwall} - t_{wall}\right)}{2} + 1.5\cdot M_{EHWall} = 43.28\cdot\frac{kip\cdot ft}{ft}$$

Load Case III

Factored vertical force at the base of the wall

$$F_{VWallLC3StrI} := 1.25 \cdot (DC_{Sup} + DC_{backwall} + DC_{wall}) + 1.5DW_{Sup} + 1.75R_{LLWallMax}$$

$$F_{VWallLC3StrI} = 29.861 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC3StrI} := 1.5 \cdot P_{EHWall} = 4.693 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC3StrI} := 0.9 \cdot DC_{backwall} \cdot \frac{(t_{backwall} - t_{wall})}{2} \ldots$$
$$+ (1.25 \cdot DC_{Sup} + 1.5 \cdot DW_{Sup} + 1.75 \cdot R_{LLWallMax}) \cdot \left(l_{brtowall} - \frac{t_{wall}}{2}\right) \ldots$$
$$+ 1.5 \cdot M_{EHWall}$$

$$M_{uWallLC3StrI} = 56.969 \cdot \frac{kip \cdot ft}{ft}$$

Load Case IV

Factored vertical force at the base of the wall

$$F_{VWallLC4StrI} := 1.25 \cdot (DC_{Sup} + DC_{backwall} + DC_{wall}) + 1.5DW_{Sup} = 20.012 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC4StrI} := 1.5 \cdot P_{EHWall} + 1.75 \cdot P_{LSWall} + 0.5TU = 6.57 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC4StrI} := 0.9 \cdot DC_{backwall} \cdot \frac{(t_{backwall} - t_{wall})}{2} + (1.25 \cdot DC_{Sup} + 1.5 \cdot DW_{Sup}) \cdot \left(l_{brtowall} - \frac{t_{wall}}{2}\right) \ldots$$
$$+ 1.5 \cdot M_{EHWall} + 1.75 \cdot M_{LSWall} + 0.5 \cdot TU \cdot h_{wall}$$

$$M_{uWallLC4StrI} = 73.864 \cdot \frac{kip \cdot ft}{ft}$$

Service I

Service I = 1.0DC + 1.0DW + 1.0LL + 1.0BR + 1.0WS + 1.0WL + 1.0EH + 1.0EV + 1.0LS + 1.0TU

Load Case I

Factored vertical force at the base of the wall

$$F_{VWallLC1SerI} := DC_{backwall} + DC_{wall} = 9.288 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC1SerI} := P_{EHWall} = 3.129 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC1SerI} := DC_{backwall} \cdot \frac{(t_{backwall} - t_{wall})}{2} + M_{EHWall}$$

$$M_{uWallLC1SerI} = 28.535 \cdot \frac{kip \cdot ft}{ft}$$

<u>Load Case III</u>

Factored vertical force at the base of the wall

$$F_{VWallLC3SerI} := (DC_{Sup} + DC_{backwall} + DC_{wall}) + DW_{Sup} + R_{LLWallMax}$$

$$F_{VWallLC3SerI} = 21.46 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC3SerI} := P_{EHWall} = 3.129 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC3SerI} := DC_{backwall} \cdot \frac{(t_{backwall} - t_{wall})}{2} \dots$$
$$+ (DC_{Sup} + DW_{Sup} + R_{LLWallMax}) \cdot \left(l_{brtowall} - \frac{t_{wall}}{2}\right) \dots$$
$$+ M_{EHWall}$$

$$M_{uWallLC3SerI} = 37.664 \cdot \frac{kip \cdot ft}{ft}$$

<u>Load Case IV</u>

Factored vertical force at the base of the wall

$$F_{VWallLC4SerI} := (DC_{Sup} + DC_{backwall} + DC_{wall}) + 1.0DW_{Sup}$$

$$F_{VWallLC4SerI} = 15.832 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the abutment wall

$$V_{uWallLC4SerI} := P_{EHWall} + P_{LSWall} + TU = 4.399 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the abutment wall

$$M_{uWallLC4SerI} := DC_{backwall} \cdot \frac{(t_{backwall} - t_{wall})}{2} + (1.0 \cdot DC_{Sup} + 1.0 \cdot DW_{Sup}) \cdot \left(l_{brtowall} - \frac{t_{wall}}{2}\right) \dots$$
$$+ 1.0 \cdot M_{EHWall} + 1.0 \cdot M_{LSWall} + 1.0 \cdot TU \cdot h_{wall}$$

$$M_{uWallLC4SerI} = 50.795 \cdot \frac{kip \cdot ft}{ft}$$

Summary of the Forces and Moments at the Base of the Abutment Wall

Factored vertical force, F_{VWall} (kip/ft)

	Strength I	Service I
LC I	11.61	9.29
LC III	29.86	21.46
LC IV	20.01	15.83

Factored shear force parallel to the transverse axis of the abutment wall, V_{uWall} (kip/ft)

	Strength I	Service I
LC I	4.69	3.13
LC III	4.69	3.13
LC IV	6.57	4.40

Factored moment about the longitudinal axis of the abutment wall, M_{uWall} (kip·ft/ft)

	Strength I	Service I
LC I	43.28	28.53
LC III	56.97	37.66
LC IV	73.86	50.80

The forces and moments presented in the tables above are used for the structural design presented in Step 3.8. As per the MDOT practice reflected in the Bridge Design System (BDS), the MDOT legacy software, the lateral earth load within the EPS backfill zone is excluded. The following tables present the forces and moments at the base of the abutment wall after excluding the lateral earth load within the EPS backfill zone. This summary is presented for informational purposes only.

Factored vertical force, F_{VWall} (kip/ft)

	Strength I	Service I
LC I	11.61	9.29
LC III	29.86	21.46
LC IV	20.01	15.83

Factored shear force parallel to the transverse axis of the abutment wall, V_{uWall} (kip/ft)

	Strength I	Service I
LC I	4.29	2.86
LC III	4.29	2.86
LC IV	6.31	4.41

Factored moment about the longitudinal axis of the abutment wall, M_{uWall} (kip·ft/ft)

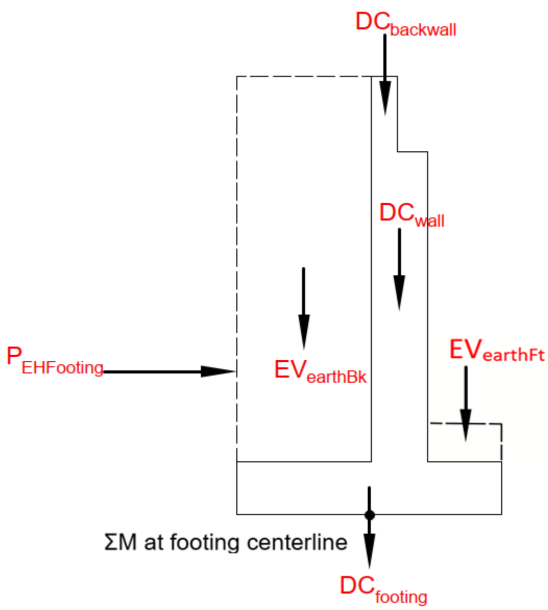
	Strength I	Service I
LC I	40.87	26.93
LC III	54.56	36.06
LC IV	73.89	54.06

Forces and Moments at the Base of the Footing

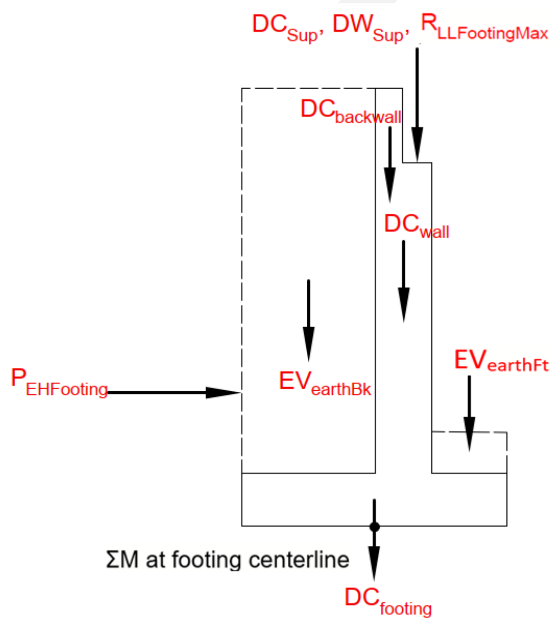
LRFD 3.6.2.1

Load Cases I, III, and IV are considered. In addition to all the loads considered for the abutment wall, weight of soil (earth load) on the footing toe and heel and live load surcharge on the heel are considered.

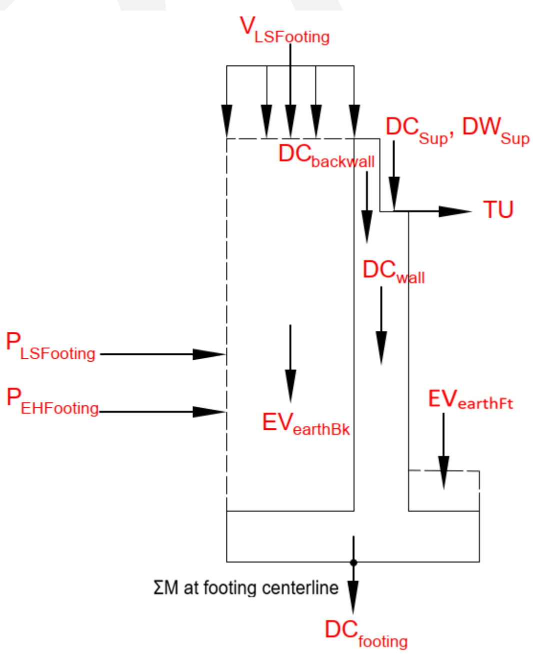
The dynamic load allowance is excluded from the live load for foundation components located entirely below ground level.



LC I



LC III



LC IV

Strength I

Strength I = 1.25DC + 1.5DW + 1.75LL + 1.75BR + 1.5EH + 1.35EV + 1.75LS + 0.5TU

<u>Load Case I</u>

Factored vertical force at the base of the footing

$$F_{VFtLC1StrI} := 1.25\cdot\left(DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.35\cdot\left(EV_{earthBk} + EV_{earthFt}\right) = 29.781\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC1StrI} := 1.5\cdot P_{EHFooting} = 6.555\cdot\frac{kip}{ft}$$

The vertical earth load of the backfill soil reduces the critical moment about the footing longitudinal axis. This requires the use of the minimum load factor of 1.0 for EV instead of the factor 1.35 in the Strength I combination.

LRFD 3.4.1

The same is done for the moment calculated about the footing longitudinal axis for all the load cases and limit states.

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC1StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \dots$$
$$+ 1.5\cdot M_{EHFooting} + 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + 1.35\cdot EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC1StrI} = 38.136\cdot\frac{kip\cdot ft}{ft}$$

<u>Load Case III</u>

Factored vertical force at the base of the footing

$$F_{VFtLC3StrI} := 1.25\cdot\left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.5DW_{Sup} + 1.75R_{LLFootingMax} \dots$$
$$+ 1.35\cdot\left(EV_{earthBk} + EV_{earthFt}\right)$$

$$F_{VFtLC3StrI} = 47.733\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC3StrI} := 1.5\cdot P_{EHFooting} = 6.555\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \dots$$
$$+ \left(1.25\cdot DC_{Sup} + 1.5\cdot DW_{Sup} + 1.75\cdot R_{LLFootingMax}\right)\cdot\left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \dots$$
$$+ 1.5\cdot M_{EHFooting} + 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + 1.35\cdot EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC3StrI} = 30.656\cdot\frac{kip\cdot ft}{ft}$$

Load Case IV

Factored vertical force at the base of the footing

$$F_{VFtLC4StrI} := 1.25\cdot\left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.5DW_{Sup} \cdots + 1.35\cdot\left(EV_{earthFt} + EV_{earthBk}\right) + 1.75V_{LSFooting}$$

$$F_{VFtLC4StrI} = 39.863\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC4StrI} := 1.5\cdot P_{EHFooting} + 1.75\cdot P_{LSFooting} + 0.5TU = 8.809\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots + \left(1.25\cdot DC_{Sup} + 1.5\cdot DW_{Sup}\right)\cdot\left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \cdots + 1.5\cdot M_{EHFooting} + 1.75M_{LSFooting} + 1.75V_{LSFooting}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) \cdots + 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + 1.35\cdot EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right) \cdots + 0.5\cdot TU\cdot\left(h_{wall} + t_{footing}\right)$$

$$M_{uFtLC4StrI} = 57.133\cdot\frac{kip\cdot ft}{ft}$$

Service I

Service I = 1.0DC + 1.0DW + 1.0LL + 1.0BR + 1.0WS + 1.0WL + 1.0EH + 1.0EV + 1.0LS + 1.0TU

Load Case I

Factored vertical force at the base of the footing

$$F_{VFtLC1SerI} := DC_{backwall} + DC_{wall} + DC_{footing} + EV_{earthBk} + EV_{earthFt} = 23.198\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC1SerI} := P_{EHFooting} = 4.37\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC1SerI} := DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots + M_{EHFooting} + EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC1SerI} = 16.982\cdot\frac{kip\cdot ft}{ft}$$

Load Case III

Factored vertical force at the base of the footing

$$F_{VFtLC3SerI} := DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing} + DW_{Sup} + R_{LLFootingMax} \cdots + \left(EV_{earthFt} + EV_{earthBk}\right)$$

$$F_{VFtLC3SerI} = 35.199 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC3SerI} := P_{EHFooting} = 4.37 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3SerI} := DC_{backwall} \cdot \left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall} \cdot \left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots + \left(DC_{Sup} + DW_{Sup} + R_{LLFootingMax}\right) \cdot \left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \cdots + M_{EHFooting} + EV_{earthBk} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt} \cdot \left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC3SerI} = 11.982 \cdot \frac{kip \cdot ft}{ft}$$

Load Case IV

Factored vertical force at the base of the footing

$$F_{VFtLC4SerI} := DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing} + DW_{Sup} \cdots + \left(EV_{earthFt} + EV_{earthBk}\right) + V_{LSFooting}$$

$$F_{VFtLC4SerI} = 30.702 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC4SerI} := P_{EHFooting} + P_{LSFooting} + TU = 5.856 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4SerI} := DC_{backwall} \cdot \left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall} \cdot \left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots + \left(DC_{Sup} + DW_{Sup}\right) \cdot \left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) + M_{EHFooting} \cdots + EV_{earthBk} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt} \cdot \left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right) \cdots + V_{LSFooting} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + M_{LSFooting} + TU \cdot \left(h_{wall} + t_{footing}\right)$$

$$M_{uFtLC4SerI} = 31.183 \cdot \frac{kip \cdot ft}{ft}$$

Summary of Forces and Moments at the Base of the Footing

Factored vertical force, F_{VFt} (kip/ft)

	Strength I	Service I
LC I	29.78	23.20
LC III	47.73	35.20
LC IV	39.86	30.70

Factored shear force parallel to the transverse axis of the footing, V_{uFt} (kip/ft)

	Strength I	Service I
LC I	6.55	4.37
LC III	6.55	4.37
LC IV	8.81	5.86

Factored moment about the longitudinal axis of the footing, M_{uFt} (kip·ft/ft)

	Strength I	Service I
LC I	38.14	16.98
LC III	30.66	11.98
LC IV	57.13	31.18

The forces and moments presented in the tables above are used for the designs presented in Step 3.6. and 3.9. As per the MDOT practice reflected in the Bridge Design System (BDS), the MDOT legacy software, the lateral earth load within the EPS backfill zone is excluded. The following tables present the forces and moments at the base of the abutment wall after excluding the lateral earth load within the EPS backfill zone. This summary is presented for informational purposes only.

Factored vertical force, F_{VFt} (kip/ft)

	Strength I	Service I
LC I	29.78	23.20
LC III	47.73	35.20
LC IV	39.86	30.70

Factored shear force parallel to the transverse axis of the footing, V_{uFt} (kip/ft)

	Strength I	Service I
LC I	6.15	4.10
LC III	6.15	4.10
LC IV	8.55	5.87

Factored moment about the longitudinal axis of the footing, M_{uFt} (kip·ft/ft)

	Strength I	Service I
LC I	34.52	14.57
LC III	27.04	9.57
LC IV	56.37	34.47

Step 3.6 Geotechnical Design of the Footing

Description

This step presents the geotechnical design of a spread footing considering the following strength and serviceability limit states:

LRFD 10.6.1.1

1. bearing resistance – strength limit state
2. settlement – service limit state
3. sliding resistance – strength limit state
4. load eccentricity (overturning) – strength limit state

The evaluation of structural resistance of the footing (internal stability) is presented later in Step 3.9.

Page Contents

Summary of Forces and Moments at the Base of the Footing

As per the MDOT practice reflected in the Bridge Design System (BDS), the MDOT legacy software, the lateral earth load within the EPS backfill zone is excluded. The following tables present the forces and moments at the base of the footing after including the lateral earth load within the EPS backfill zone. The forces and moments presented in these tables are used for the designs presented in Step 3.6. and 3.9.

Factored vertical force, F_{VFt} (kip/ft)

	Strength I	Service I
LC I	29.78	23.20
LC III	47.73	35.20
LC IV	39.86	30.70

Factored shear force parallel to the transverse axis of the footing, V_{uFt} (kip/ft)

	Strength I	Service I
LC I	6.55	4.37
LC III	6.55	4.37
LC IV	8.81	5.86

Factored moment about the longitudinal axis of the footing, M_{uFt} (kip·ft/ft)

	Strength I	Service I
LC I	38.14	16.98
LC III	30.66	11.98
LC IV	57.13	31.18

Bearing Resistance Check

AASHTO LRFD 10.6.1.3 states for eccentrically loaded footing, a reduced effective area shall be used in geotechnical design for bearing resistance or settlement. The point of load application shall be at the centroid of the reduced area. **LRFD 10.6.1.3**

In MDOT practice, the abutment footing pressures at the toe and heel, and the average pressure under different load cases and limit states are provided to Geotechnical Service Section for verification.

Both methods for checking bearing strength are presented in this step.

Load Case I, Strength I

Factored vertical force

$$F_{VFtLC1StrI} = 29.781 \cdot \frac{kip}{ft}$$

Factored moment about footing longitudinal axis

$$M_{uFtLC1StrI} = 38.136 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC1StrI}}{F_{VFtLC1StrI}} = 1.281\ ft$$

LRFD method

For eccentrically loaded footings, a reduced effective footing width shall be used in geotechnical design for settlement or bearing resistance. **LRFD 10.6.1.3**

Effective footing width

$$B_{eff} := B_{footing} - 2 \cdot e_B = 10.939\ ft$$

LRFD Eq. 10.6.1.3-1

Footing bearing pressure

$$q_{bearing\_LC1} := \frac{F_{VFtLC1StrI}}{B_{eff}} = 2.722 \cdot ksf$$

MDOT method

Average bearing pressure

$$q_{avgLC1} := \frac{F_{VFtLC1StrI}}{B_{footing}} = 2.206 \cdot ksf$$

Toe bearing pressure

$$q_{toeLC1} := \frac{F_{VFtLC1StrI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.461 \cdot ksf$$

Heel bearing pressure

$$q_{heelLC1} := \frac{F_{VFtLC1StrI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 0.95 \cdot ksf$$

Load Case III, Strength I

Factored vertical force

$$F_{VFtLC3StrI} = 47.733 \cdot \frac{kip}{ft}$$

Factored moment about footing longitudinal axis

$$M_{uFtLC3StrI} = 30.656 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC3StrI}}{F_{VFtLC3StrI}} = 0.642\ ft$$

LRFD method

Effective footing width

$$B_{eff} := B_{footing} - 2 \cdot e_B = 12.216\ ft$$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure

$$q_{bearing\_LC3} := \frac{F_{VFtLC3StrI}}{B_{eff}} = 3.908 \cdot ksf$$

MDOT method

Average bearing pressure

$$q_{avgLC3} := \frac{F_{VFtLC3StrI}}{B_{footing}} = 3.536 \cdot ksf$$

Toe bearing pressure

$$q_{toeLC3} := \frac{F_{VFtLC3StrI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 4.545 \cdot ksf$$

Heel bearing pressure

$$q_{heelLC3} := \frac{F_{VFtLC3StrI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 2.527 \cdot ksf$$

Load Case IV, Strength I

Factored vertical force

$$F_{VFtLC4StrI} = 39.863 \cdot \frac{kip}{ft}$$

Factored moment about footing longitudinal axis

$$M_{uFtLC4StrI} = 57.133 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI}}{F_{VFtLC4StrI}} = 1.433\ ft$$

LRFD method

Effective footing width $B_{eff} := B_{footing} - 2 \cdot e_B = 10.634\ ft$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure $q_{bearing\_LC4} := \frac{F_{VFtLC4StrI}}{B_{eff}} = 3.749 \cdot ksf$

MDOT method

Average bearing pressure $q_{avgLC4} := \frac{F_{VFtLC4StrI}}{B_{footing}} = 2.953 \cdot ksf$

Toe bearing pressure $q_{toeLC4} := \frac{F_{VFtLC4StrI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 4.834 \cdot ksf$

Heel bearing pressure $q_{heelLC4} := \frac{F_{VFtLC4StrI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 1.072 \cdot ksf$

Load Case I, Service I

Factored vertical force $F_{VFtLC1SerI} = 23.198 \cdot \frac{kip}{ft}$

Factored moment about footing longitudinal axis $M_{uFtLC1SerI} = 16.982 \cdot \frac{kip \cdot ft}{ft}$

Eccentricity in the footing width direction $e_B := \frac{M_{uFtLC1SerI}}{F_{VFtLC1SerI}} = 0.732\ ft$

LRFD method

Effective footing width $B_{eff} := B_{footing} - 2 \cdot e_B = 12.036\ ft$ **LRFD Eq. 10.6.1.3-1**

Footing bearing pressure $q_{bearing\_LC1SerI} := \frac{F_{VFtLC1SerI}}{B_{eff}} = 1.927 \cdot ksf$

MDOT method

Average bearing pressure $q_{avgLC1SerI} := \frac{F_{VFtLC1SerI}}{B_{footing}} = 1.718 \cdot ksf$

Toe bearing pressure $q_{toeLC1SerI} := \frac{F_{VFtLC1SerI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 2.277 \cdot ksf$

Heel bearing pressure $q_{heelLC1SerI} := \frac{F_{VFtLC1SerI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 1.159 \cdot ksf$

Load Case III, Service I

Factored vertical force $F_{VFtLC3SerI} = 35.199 \cdot \frac{kip}{ft}$

Factored moment about footing longitudinal axis $M_{uFtLC3SerI} = 11.982 \cdot \frac{kip \cdot ft}{ft}$

Eccentricity in the footing width direction $e_B := \frac{M_{uFtLC3SerI}}{F_{VFtLC3SerI}} = 0.34\ ft$

LRFD method

Effective footing width

$B_{eff} := B_{footing} - 2 \cdot e_B = 12.819 \text{ ft}$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure

$q_{bearing\_LC3SerI} := \frac{F_{VFtLC3SerI}}{B_{eff}} = 2.746 \cdot ksf$

MDOT method

Average bearing pressure

$q_{avgLC3SerI} := \frac{F_{VFtLC3SerI}}{B_{footing}} = 2.607 \cdot ksf$

Toe bearing pressure

$q_{toeLC3SerI} := \frac{F_{VFtLC3SerI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.002 \cdot ksf$

Heel bearing pressure

$q_{heelLC3SerI} := \frac{F_{VFtLC3SerI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 2.213 \cdot ksf$

Load Case IV, Service I

Factored vertical force

$F_{VFtLC4SerI} = 30.702 \cdot \frac{kip}{ft}$

Factored moment about footing longitudinal axis

$M_{uFtLC4SerI} = 31.183 \cdot \frac{kip \cdot ft}{ft}$

Eccentricity in the footing width direction

$e_B := \frac{M_{uFtLC4SerI}}{F_{VFtLC4SerI}} = 1.016 \text{ ft}$

LRFD method

Effective footing width

$B_{eff} := B_{footing} - 2 \cdot e_B = 11.469 \text{ ft}$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure

$q_{bearing\_LC4SerI} := \frac{F_{VFtLC4SerI}}{B_{eff}} = 2.677 \cdot ksf$

MDOT method

Average bearing pressure

$q_{avgLC4SerI} := \frac{F_{VFtLC4SerI}}{B_{footing}} = 2.274 \cdot ksf$

Toe bearing pressure

$q_{toeLC4SerI} := \frac{F_{VFtLC4SerI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.301 \cdot ksf$

Heel bearing pressure

$q_{heelLC4SerI} := \frac{F_{VFtLC4SerI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 1.248 \cdot ksf$

Summary

<u>LRFD method</u>

The controlling bearing pressure under strength limit states

$$q_b := \max(q_{bearing\_LC1}, q_{bearing\_LC3}, q_{bearing\_LC4}) = 3.908 \cdot ksf$$

The controlling bearing pressure needs to be checked with the factored bearing resistance of the soil provided by the geotechnical engineer.

<u>MDOT method</u>

Bearing pressures (in psf) are shown in the the table below.

	Toe (Service I)	Avg (Serivce I)	Heel (Service I)	Toe (Strength I)	Avg (Strength I)	Heel (Strength I)
LC I	2277	1718	1159	3461	2206	950
LC III	3002	2607	2213	4545	3536	2527
LC IV	3301	2274	1248	4834	2953	1072

The table is provided to Geotechnical Service Section for verification of the bearing resistance. If the bearing pressure exceeds the bearing strength of the soil, the size of the footing needs to be increased. See BDM 7.03.02.G for more information.

Settlement Check

BDM 7.03.02G 2b

The geotechnical engineer uses the controlling bearing pressure from the service limit state to check if the foundation total settlement is less than 1.5 in., the allowable limit.

For LRFD method, the controlling bearing pressure for settlement analysis is

$$q_{b\_settlement} := \max(q_{bearing\_LC1SerI}, q_{bearing\_LC3SerI}, q_{bearing\_LC4SerI}) = 2.746 \cdot ksf$$

The geotechnical engineer uses this controlling bearing pressure to calculate the foundation total settlement.

For MDOT method, the bearing pressures under service limit state are provided to the Geotechnical Service Section to calculate the settlement.

Note: Besides the total settlement, considerations should be given to prevent the differential settlement between the abutments and pier from exceeding the tolerable differential settlement limit. Differential settlement limits are given in the *Steel Plate Girder Example.*

Sliding Resistance Check

LRFD 10.6.3.4

Spread footings must be designed to resist lateral loads without sliding. The sliding resistance of a footing on cohesionless soil is a function of the normal force and the interface friction between the foundation and the soil.

The geotechnical engineer should provide a coefficient of sliding resistance (μ) for design calculations. MDOT typically uses a sliding resistance coefficient of 0.5 for cast-in-place concrete footings. Consult the geotechnical engineer to identify the most suitable coefficient for a specific design.

Coefficient of sliding resistance $\mu := 0.5$

The strength limit states are used for this check. Since the resistance is proportional to the vertical loads, the following conditions are used.

- Live load on the bridge is excluded.
- Minimum load factors are used for all vertical loads.
- Maximum load factors are used for the loads that contribute to horizontal sliding forces.

Since DW is the future wearing surface load, it is excluded from all load combinations.

Load Case I

Factored shear force parallel to the transverse axis of the footing $\quad V_{uFtLC1StrI} = 6.555 \cdot \frac{kip}{ft}$

Factored sliding force $\quad V_{sliding} := V_{uFtLC1StrI} = 6.555 \cdot \frac{kip}{ft}$

Minimum vertical load

$$F_{VFtLC1StrIMin} := 0.9 \cdot \left(DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.0 \cdot \left(EV_{earthBk} + EV_{earthFt}\right) = 21.662 \cdot \frac{kip}{ft}$$

Resistance factor for sliding $\quad \phi_\tau := 0.8 \quad$ **BDM 7.03.02.F, LRFD Table 10.5.5.5.2-1**

Sliding resistance $\quad V_{resistance} := \phi_\tau \cdot \mu \cdot F_{VFtLC1StrIMin} = 8.665 \cdot \frac{kip}{ft}$

Check if $V_{resistance} > V_{sliding}$ $\quad Check := if\left(V_{resistance} > V_{sliding}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Load Case III

Factored shear force parallel to the transverse axis of the footing $\quad V_{uFtLC3StrI} = 6.555 \cdot \frac{kip}{ft}$

Factored sliding force $\quad V_{sliding} := V_{uFtLC3StrI} = 6.555 \cdot \frac{kip}{ft}$

When calculating the minimum vertical force for sliding and eccentric load limitation checks, the live load on the superstructure is excluded to develop a conservative design.

Minimum vertical load

$$F_{VFtLC3StrIMin\_noLL} := 0.9 \cdot \left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) \dots + 1.0 \cdot \left(EV_{earthBk} + EV_{earthFt}\right)$$

$$F_{VFtLC3StrIMin\_noLL} = 26.754 \cdot \frac{kip}{ft}$$

Sliding resistance $\quad V_{resistance} := \phi_\tau \cdot \mu \cdot F_{VFtLC3StrIMin\_noLL} = 10.702 \cdot \frac{kip}{ft}$

Check if $V_{resistance} > V_{sliding}$ $\quad Check := if\left(V_{resistance} > V_{sliding}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Load Case IV

There are two cases that need to be considered: without and with live load surcharge.

Without live load surcharge:

Factored shear force parallel to the transverse axis of the footing $\quad V_{uFtLC4StrI} = 8.809 \cdot \frac{kip}{ft}$

Factored sliding force $\quad V_{sliding} := V_{uFtLC4StrI} - 1.75 P_{LSFooting} = 6.693 \cdot \frac{kip}{ft}$

Minimum vertical load

$$F_{VFtLC4StrIMin\_noLS} := 0.9 \cdot \left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) \dots + 1.0 \cdot \left(EV_{earthBk} + EV_{earthFt}\right)$$

$$F_{VFtLC4StrIMin\_noLS} = 26.754 \cdot \frac{kip}{ft}$$

Sliding resistance

$$V_{resistance} := \phi_{\tau} \cdot \mu \cdot F_{VFtLC4StrIMin\_noLS} = 10.702 \cdot \frac{kip}{ft}$$

Check if $V_{resistance} > V_{sliding}$

$$Check := if\left(V_{resistance} > V_{sliding}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

With live load surcharge:

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC4StrI} = 8.809 \cdot \frac{kip}{ft}$$

Factored sliding force

$$V_{sliding} := V_{uFtLC4StrI} = 8.809 \cdot \frac{kip}{ft}$$

Minimum vertical load

$$F_{VFtLC4StrIMin} := 0.9 \cdot \left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) \dots + 1.0 \cdot \left(EV_{earthBk} + EV_{earthFt}\right) + 1.75 V_{LSFooting}$$

$$F_{VFtLC4StrIMin} = 28.434 \cdot \frac{kip}{ft}$$

Sliding resistance

$$V_{resistance} := \phi_{\tau} \cdot \mu \cdot F_{VFtLC4StrIMin} = 11.374 \cdot \frac{kip}{ft}$$

Check if $V_{resistance} > V_{sliding}$

$$Check := if\left(V_{resistance} > V_{sliding}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Eccentric Load Limitation (Overturning) Check

The eccentricity of loading at the strength limit state, evaluated based on factored loads, shall not exceed one-sixth of the corresponding dimension measured from the centerline of the footing for stability.

LRFD 10.6.3.3

The eccentricity in the abutment length direction is not of a concern. The eccentricity in the abutment width direction at Strength I limit state is evaluated.

Load Case I

Minimum vertical load

$$F_{VFtLC1StrIMin} = 21.662 \cdot \frac{kip}{ft}$$

Maximum moment about the longitudinal axis of the footing

$$M_{uFtLC1StrI} = 38.136 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC1StrI}}{F_{VFtLC1StrIMin}} = 1.761\ ft$$

1/6 of footing width

$$\frac{B_{footing}}{6} = 2.25\ ft$$

Check eccentric load limitation

$$Check := if\left(e_B < \frac{B_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Load Case III

There are two cases that need to be considered: without and with live load.

Without live load:

Minimum vertical force

$$F_{VFtLC3StrIMin\_noLL} = 26.754 \cdot \frac{kip}{ft}$$

Moment about the longitudinal axis of the footing (with live load)

$$M_{uFtLC3StrI} = 30.656 \cdot \frac{kip \cdot ft}{ft}$$

Moment about the longitudinal axis of the footing (without live load)

$$M_{uFtLC3StrI\_noLL} := M_{uFtLC3StrI} - (1.75 \cdot R_{LLFootingMax}) \cdot \left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) = 34.635 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC3StrI\_noLL}}{F_{VFtLC3StrIMin\_noLL}} = 1.295\ ft$$

Check eccentric load limitation

$$Check := if\left(e_B < \frac{B_{footing}}{6}, "OK", "Not\ OK"\right) = "OK"$$

With live load:

Minimum vertical force

$$F_{VFtLC3StrIMin} := F_{VFtLC3StrIMin\_noLL} + 1.75 R_{LLFootingMax}$$

$$F_{VFtLC3StrIMin} = 36.304 \cdot \frac{kip}{ft}$$

Moment about the longitudinal axis of the footing

$$M_{uFtLC3StrI} = 30.656 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC3StrI}}{F_{VFtLC3StrIMin}} = 0.844\ ft$$

Check eccentric load limitation

$$Check := if\left(e_B < \frac{B_{footing}}{6}, "OK", "Not\ OK"\right) = "OK"$$

Load Case IV

There are two cases that need to be considered: without and with live load surcharge.

Without live load surcharge:

Minimum vertical force

$$F_{VFtLC4StrIMin\_noLS} = 26.754 \cdot \frac{kip}{ft}$$

Moment about the longitudinal axis of the footing (with live load surcharge)

$$M_{uFtLC4StrI} = 57.133 \cdot \frac{kip \cdot ft}{ft}$$

Moment about the longitudinal axis of the footing (without live load surcharge)

$$M_{uFtLC4StrI\_noLS} := M_{uFtLC4StrI} - 1.75 V_{LSFooting} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) \dots$$
$$+ (-1.75) \cdot P_{LSFooting} \cdot \frac{(h_{backwall} + h_{wall} + t_{footing})}{2}$$

$$M_{uFtLC4StrI\_noLS} = 38.891 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI\_noLS}}{F_{VFtLC4StrIMin\_noLS}} = 1.454\ ft$$

Check eccentric load limitation

$$Check := if\left(e_B < \frac{B_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

With live load surcharge:

Minimum vertical force

$$F_{VFtLC4StrIMin} = 28.434 \cdot \frac{kip}{ft}$$

Moment about the longitudinal axis of the footing

$$M_{uFtLC4StrI} = 57.133 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI}}{F_{VFtLC4StrIMin}} = 2.009\ ft$$

Check eccentric load limitation

$$Check := if\left(e_B < \frac{B_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Step 3.7 Backwall Design

Description

Please refer to the design calculations and details presented in Step 2.7. The backwall forces and moments used in Step 2.7. are not impacted by the use of EPS blocks as the backfill material since the EPS blocks are located below the backwall.

Step 3.8 Abutment Wall Design

Description

This step presents the design of the abutment wall.

Page Contents

Forces and Moments at the Base of the Abutment Wall

A summary of load effects under different load cases and limit states from Step 3.5 is listed in the following tables.

Factored vertical force, F_{VWall} (kip/ft)

	Strength I	Service I
LC I	11.61	9.29
LC III	29.86	21.46
LC IV	20.01	15.83

Factored shear force parallel to the transverse axis of the abutment wall, V_{uWall} (kip/ft)

	Strength I	Service I
LC I	4.69	3.13
LC III	4.69	3.13
LC IV	6.57	4.40

Factored moment about the longitudinal axis of the abutment wall, M_{uWall} (kip·ft/ft)

	Strength I	Service I
LC I	43.28	28.53
LC III	56.97	37.66
LC IV	73.86	50.80

Design for Flexure

By examining the forces and moments at the base of the abutment wall in the summary tables, Load Case IV under Strength I limit state is identified as the governing load case for the flexural design.

Moment demand at the base of the wall $\quad M_{DemandWall} := M_{uWallLC4StrI} = 73.864 \cdot \frac{kip \cdot ft}{ft}$

Flexure Resistance

LRFD 5.6.3.2

The design procedure consists of calculating the reinforcing steel area required to satisfy the moment demand and checking the selected steel area against the requirements and limitations for developing an adequate moment capacity, controlling crack width, and managing shrinkage and temperature stresses.

As a trial, select No. 8 bars. $\quad bar := 8$

Nominal diameter of a reinforcing steel bar $\quad d_{bar} := Dia(bar) = 1 \cdot in$

Cross-section area of the reinforcing steel bar on the flexural tension side $\quad A_{bar} := Area(bar) = 0.79 \cdot in^2$

The spacing of reinforcement shall not exceed 12 in. when the thickness of walls and footings is greater than 18 in.

LRFD 5.10.6

Wall thickness $\quad t_{wall} = 38 \cdot in$

Assumed spacing of reinforcement $\quad s_{bar} := 12 \cdot in$

Initial reinforcement area $\quad A_{sProvided} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.79 \cdot in^2$

Effective depth $\quad d_e := t_{wall} - Cover_{wall} = 35 \cdot in$

Resistance factor for flexure $\phi_f := 0.9$ **LRFD 5.5.4.2**

A 1-ft wide strip is selected for the design.

Width of the compression face of the member $b := 12in$

Stress block factor $\beta_1 := \min\left[\max\left[0.85 - 0.05\cdot\left(\frac{f_c - 4ksi}{ksi}\right), 0.65\right], 0.85\right] = 0.85$ **LRFD 5.6.2.2**

Solve the following quadratic equation to calculate the required A_s needed to satisfy the moment demand. Use an assumed initial A_s value to solve the quadratic equation.

Initial assumption $A_s := 1in^2$

Solve the quadratic equation for the area of steel required Given $M_{DemandWall}\cdot ft = \phi_f\cdot A_s\cdot f_y\cdot\left[d_e - \frac{1}{2}\cdot\left(\frac{A_s\cdot f_y}{0.85\cdot f_c\cdot b}\right)\right]$ **LRFD 5.6.3.2**

Required steel area $A_{sRequired} := Find(A_s) = 0.475\cdot in^2$

Check if $A_{sProvided} > A_{sRequired}$ $Check := if(A_{sProvided} > A_{sRequired}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$

Moment capacity of the section with the provided steel area

$$M_{CapacityWall} := \phi_f\cdot A_{sProvided}\cdot f_y\cdot\frac{\left[d_e - \frac{1}{2}\cdot\left(\frac{A_{sProvided}\cdot f_y}{0.85\cdot f_c\cdot b}\right)\right]}{ft}$$

$$M_{CapacityWall} = 121.672\cdot\frac{kip\cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis $c := \frac{A_{sProvided}\cdot f_y}{0.85\cdot f_c\cdot\beta_1\cdot b} = 1.82\cdot in$

Check the validity of assumption $f_s = f_y$ $Check := if\left(\frac{c}{d_e} < 0.6, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Limits for Reinforcement **LRFD 5.6.3.3**

The tensile reinforcement provided must develop a factored flexural resistance equal to the lesser of the cracking moment or 1.33 times the factored moment from the applicable strength limit state load combinations.

Flexural cracking variability factor $\gamma_1 := 1.6$ For concrete structures that are not precast segmental

Ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement $\gamma_3 := 0.67$ For ASTM615 grade 60 reinforcement

Section modulus $S_c := \frac{1}{6}\cdot b\cdot t_{wall}^2 = 2.888\times 10^3\cdot in^3$

Cracking moment $M_{cr} := \frac{\gamma_3\cdot\gamma_1\cdot f_r\cdot S_c}{ft} = 107.246\cdot\frac{kip\cdot ft}{ft}$

1.33 times the factored moment demand $1.33 \cdot M_{DemandWall} = 98.239 \cdot \frac{kip \cdot ft}{ft}$

Required moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33M_{DemandWall}, M_{cr}\right) = 98.239 \cdot \frac{kip \cdot ft}{ft}$

Check the adequacy of the section capacity $Check := if\left(M_{CapacityWall} > M_{req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions extends the service life. The width of potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stresses.

The spacing requirements for the mild steel reinforcement in the layer closest to the tension face $s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ **LRFD Eq. 5.6.7-1**

Exposure factor for Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest bar $d_c := Cover_{wall} = 3 \cdot in$

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer closest to the tension face $\beta_s := 1 + \frac{d_c}{0.7\left(t_{wall} - d_c\right)} = 1.122$

The position of the section's neutral axis is determined through an iterative process to calculate the actual stress in the reinforcement. This process starts with an assumed position of the neutral axis.

Assumed distance from the extreme compression fiber to the neutral axis $x := 6 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvided} \cdot \left(d_e - x\right)$

Position of the neutral axis $x_{na} := Find(x) = 5.568 \cdot in$

Tensile force in the reinforcing steel due to service limit state moment $T_s := \frac{M_{uWallLC4SerI}}{d_e - \frac{x_{na}}{3}} \cdot ft = 18.4 \cdot kip$

Stress in the reinforcing steel due to service limit state moment $f_{ss1} := \frac{T_s}{A_{sProvided}} = 23.279 \cdot ksi$

f_{ss} not to exceed $0.6f_y$ $f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 23.279 \cdot ksi$

Required reinforcement spacing $s_{barRequred} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 20.789 \cdot in$

Check if the spacing provided < the required spacing $Check := if\left(s_{bar} < s_{barRequred}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Shrinkage and Temperature Reinforcement Requirement

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the backwall.

For bars, the area of reinforcement per foot (A_s), on each face and in each direction, shall satisfy

$$A_S \geq \frac{1.3bh}{2(b+h)f_y}$$

LRFD 5.10.6

and

$$0.11in^2 \leq A_S \leq 0.6in^2$$

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := \min\left[\left[0.60\frac{in^2}{ft}\right], \max\left[\left[0.11\frac{in^2}{ft}\right], \left[\frac{1.3\cdot h_{wall}\cdot t_{wall}\cdot \frac{kip}{in\cdot ft}}{2(h_{wall}+t_{wall})\cdot f_y}\right]\right]\right]\cdot ft = 0.349\cdot in^2$$

Check if the provided area of steel > the required area of shrinkage and temperature steel

$$Check := if\left(A_{sProvided} > A_{shrink.temp}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Design for Shear

By examining the loads in the summary tables, Load Case IV under Strength I limit state is identified as the governing load case for shear design.

Maximum factored shear force at the base of the abutment wall

$$V_{uWallLC4StrI} = 6.57\cdot\frac{kip}{ft}$$

Effective width of the section

$$b_v := b = 12\cdot in$$

Depth of equivalent rectangular stress block

$$a := \frac{A_{sProvided}\cdot f_y}{0.85\cdot f_c\cdot b} = 1.549\cdot in$$

Effective shear depth

$$d_v := \max\left(d_e - \frac{a}{2}, 0.9\cdot d_e, 0.72\cdot t_{wall}\right) = 34.225\cdot in$$

LRFD 5.7.2.8

Since there is no transverse reinforcement in the wall and the overall depth of the wall is greater than 16 in., the simplified procedure in LRFD 5.7.3.4.1 can not be used. The general procedure outlined in LRFD 5.7.3.4.2 is used for the shear design of the wall.

The factored N_u, V_u, and M_u at the critical section for shear, which is at a distance d_v from the base of the abutment wall, are calculated. The critical section is located in the wall segment with EPS backfill.

Factored axial force at the critical section for shear, taking as negative if compression

$$N_{uWallShear} := -\left[1.25\cdot\left(DC_{Sup} + DC_{backwall} + DC_{wall} - d_v\cdot t_{wall}\cdot W_c\right) \dots + 1.5DW_{Sup}\right]$$

$$N_{uWallShear} = -18.318\cdot\frac{kip}{ft}$$

The lateral earth load component at the critical section for shear consists of three parts. Part 1 is the lateral load from the soil above the EPS blocks, which is the same as that calculated in Step 3.4. Part 2 is the lateral load from the EPS due to the vertical load at the top of the EPS blocks. Part 3 is the lateral load from the soil behind the EPS blocks above the critical section for shear.

EH 1: the lateral load from the soil above the EPS blocks
EH 2: the lateral load from the EPS due to the vertical load at the top of the EPS blocks
EH 3: the lateral load from the soil behind the EPS blocks and above the critical section for shear.

$$P_{EHWall2Shear} := p_{VEPS} \cdot \left(h_{EPS} + h_{SoilBelowEPS} - d_v\right) = 1.042 \cdot \frac{kip}{ft}$$

$$P_{EHWall3Shear} := \frac{1}{2} k_{EPS} \cdot \gamma_s \cdot \left(h_{EPS} + h_{SoilBelowEPS} - d_v\right)^2 = 0.231 \cdot \frac{kip}{ft}$$

$$P_{EHWallShear} := P_{EH1} + P_{EH2} + P_{EHWall3Shear} = 2.445 \cdot \frac{kip}{ft}$$

The lateral live load surcharge at the critical section consists of two parts.

LS 1: the lateral load from the soil above the EPS blocks due to the live load surcharge
LS 2: the lateral load from the EPS blocks above the critical section due the the live load surcharge

$$P_{LSWall2Shear} := \frac{1}{10} \gamma_s \cdot h_{eq} \cdot \left(h_{EPS} + h_{SoilBelowEPS} - d_v\right) = 0.268 \cdot \frac{kip}{ft}$$

$$P_{LSWallShear} := P_{LSWall1} + P_{LSWall2Shear} = 0.828 \cdot \frac{kip}{ft}$$

Factored shear force at the critical section for shear

$$V_{uWallShear} := 1.5 \cdot P_{EHWallShear} + 1.75 \cdot P_{LSWallShear} + 0.5TU = 5.256 \cdot \frac{kip}{ft}$$

Factored moment at the critical section for shear

$$\begin{aligned} M_{uWallShear} := {} & 0.9 \cdot DC_{backwall} \cdot \frac{\left(t_{backwall} - t_{wall}\right)}{2} + \left(1.25 \cdot DC_{Sup} + 1.5 \cdot DW_{Sup}\right) \cdot \left(l_{brtowall} - \frac{t_{wall}}{2}\right) \ldots \\ & + 1.5P_{EH1} \cdot \left(\frac{1}{3} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS} - d_v\right) \ldots \\ & + 1.5P_{EH2} \cdot \frac{1}{2}\left(h_{EPS} + h_{SoilBelowEPS} - d_v\right) \ldots \\ & + 1.75P_{LSWall1} \cdot \left(\frac{1}{2} h_{SoilAboveEPS} + h_{EPS} + h_{SoilBelowEPS} - d_v\right) \ldots \\ & + 1.75 \cdot P_{LSWall2Shear} \cdot \frac{\left(h_{EPS} + h_{SoilBelowEPS} - d_v\right)}{2} + 0.5 \cdot TU \cdot \left(h_{wall} - d_v\right) \end{aligned}$$

$$M_{uWallShear} = 56.896 \cdot \frac{kip \cdot ft}{ft}$$

Check M_u since it cannot be less than $V_u d_v$

$$M_{uWallShear} := \max\left(M_{uWallShear}, V_{uWallShear} \cdot d_v\right) = 56.896 \cdot \frac{kip \cdot ft}{ft}$$

Net longitudinal tensile strain in the section at the centroid of the tension reinforcement

$$\varepsilon_s := \frac{\left(\frac{M_{uWallShear}}{d_v} + 0.5 \cdot N_{uWallShear} + V_{uWallShear}\right)}{E_s \cdot \frac{A_{sProvided}}{ft}} = 7.004 \times 10^{-4}$$

LRFD Eq. 5.7.3.4.2-4

Crack spacing parameter $s_x := d_v = 2.852\ \text{ft}$

Maximum aggregate size (in.) $a_g := 1.5$

MDOT Standard Specifications for Construction Table 902-1

Crack spacing parameter as influenced by the aggregate size

$$s_{xe} := \min\left[\left[\begin{matrix} 80in \\ \max\left(\left(\begin{matrix} 12in \\ s_x \cdot \dfrac{1.38}{a_g + 0.63} \end{matrix}\right)\right) \end{matrix}\right]\right] = 22.174 \cdot in$$

LRFD Eq. 5.7.3.4.2-7

Factor indicating the ability of diagonally cracked concrete to transmit tension and shear

$$\beta := \frac{4.8}{\left(1 + 750 \cdot \varepsilon_s\right)} \cdot \frac{51}{\left(39 + \dfrac{s_{xe}}{in}\right)} = 2.624$$

LRFD Eq. 5.7.3.4.2-2

Nominal shear resistance of concrete, V_n, is calculated as follows:

$$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_e = 60.3 \cdot kip$$

LRFD Eq. 5.7.3.3-3

$$V_{c2} := 0.25 f_c \cdot b \cdot d_e = 315 \cdot kip$$

LRFD Eq. 5.7.3.3-2

$$V_n := \min\left(V_{c1}, V_{c2}\right) = 60.31 \cdot kip$$

Resistance factor for shear $\phi_v := 0.9$

LRFD 5.5.4.2

Factored shear resistance $V_r := \phi_v \cdot V_n = 54.279 \cdot kip$

Check if the factored shear resistance is greater than the shear demand

$$Check := \text{if}\left(\frac{V_r}{ft} > V_{uWallShear}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Development Length of Reinforcement

The flexural reinforcing steel must be developed on each side of the critical section for its full development length.

LRFD 5.10.8.1.2, 5.10.8.2.1

Basic development length

$$l_{db} := 2.4 \cdot d_{bar} \cdot \frac{f_y}{\sqrt{f_c \cdot ksi}} = 6.928\ \text{ft}$$

LRFD Eq. 5.10.8.2.1a-2

Reinforcement location factor $\lambda_{rl} := 1$ No more than 12 in. concrete below

Coating factor $\lambda_{cf} := 1.5$ Epoxy coated bars with less than $3d_b$ cover

Distance from center of the bar to the nearest concrete surface

$$c_b := \frac{d_{bar}}{2} + Cover_{wall} = 3.5 \cdot in$$

Reinforcement confinement factor

$$\lambda_{rc} := \frac{d_{bar}}{c_b} = 0.286$$

Excess reinforcement factor

$$\lambda_{er} := \frac{A_{sRequired}}{A_{sProvided}} = 0.602$$

For normal weight concrete $\lambda := 1$

Required development length $l_d := l_{db} \cdot \frac{(\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = 1.786\ ft$ **LRFD Eq. 5.10.8.2.1a-1**

Since the footing thickness is 3 ft, adequate space is available for straight bars. However, the common practice is to use hooked bars set on the bottom reinforcing steel layer.

Shrinkage and Temperature Reinforcement

The following calculations check the required amount of reinforcing steel in the secondary direction to control shrinkage and temperature stresses in the backwall.

The reinforcement at the front face of the abutment wall and the horizontal reinforcement at the interior should satisfy the shrinkage and temperature reinforcement requirement. **LRFD 5.10.6**

The spacing of reinforcement shall not exceed 12 in. since the wall thickness is greater than 18 in. **LRFD 5.10.6**

MDOT practice is to use 18 in. as the maximum spacing. **BDG 5.16.01**

As a trial, select No. 6 bars. $bar := 6$

Nominal diameter of a reinforcing steel bar $d_{bST} := Dia(bar) = 0.75 \cdot in$

Cross-section area of the bar $A_{barST} := Area(bar) = 0.44 \cdot in^2$

Selected bar spacing $s_{barST} := 12 \cdot in$

Provided reinforcement area $A_{sProvidedST} := \frac{A_{barST} \cdot 12in}{s_{barST}} = 0.44 \cdot in^2$

The required minimum shrinkage and temperature reinforcement area at the abutment wall was calculated previously during flexural design.

Required shrinkage and temperature steel area $A_{shrink.temp} = 0.349 \cdot in^2$

Check if the provided area of steel > the required area of shrinkage and temperature steel $Check := if(A_{sProvidedST} > A_{shrink.temp}, "OK", "Not OK") = "OK"$

The abutment wall design presented in this step provides the following details:

- No. 8 bars at 12.0 in. spacing ($A_s = 0.79$ in.^{2}/ft) as the back face flexural reinforcement.
- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the front face vertical shrinkage and temperature reinforcement.
- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the front and back face horizontal shrinkage and temperature reinforcement.

Step 3.9 Structural Design of the Footing

Description

This step presents the structural design process for the abutment footing.

Page Contents

Forces and Moments at the Base of the Footing

A summary of load effects under different load cases and limit states from Step 3.5 is listed in the following tables.

Factored vertical force, F_{VFt} (kip/ft)

	Strength I	Service I
LC I	29.78	23.20
LC III	47.73	35.20
LC IV	39.86	30.70

Factored shear force parallel to the transverse axis of the footing, V_{uFt} (kip/ft)

	Strength I	Service I
LC I	6.55	4.37
LC III	6.55	4.37
LC IV	8.81	5.86

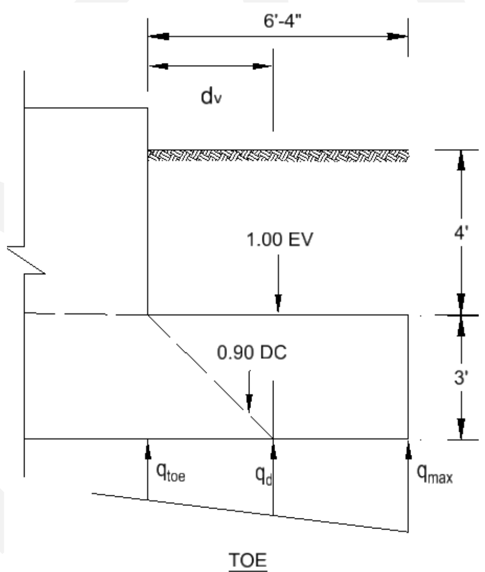
Factored moment about the longitudinal axis of the footing, M_{uFt} (kip·ft/ft)

	Strength I	Service I
LC I	38.14	16.98
LC III	30.66	11.98
LC IV	57.13	31.18

Note: In this example, the length of the footing and the abutment wall are 65.75 ft and 63.75 ft, respectively. Since the cantilevered length of the footing in the longitudinal direction is limited to 1 ft on each side, the shear and moment acting on the footing in the longitudinal direction are small and do not require flexural and shear designs.

Toe Design

A summary of dimensions and loads for the design is shown in the following figure.



For structural design of an eccentrically loaded foundation, a triangular or trapezoidal contact stress distribution based on the factored loads shall be used.

LRFD 10.6.5

By examining the summary tables of the forces and moments at the base of the footing presented in Step 3.5, Load Case IV under Strength I limit state is identified as the governing load case for the flexural and shear design of the toe.

$F_{VFtLC4StrI} = 39.863 \cdot \frac{kip}{ft}$ $\quad$ $M_{uFtLC4StrI} = 57.133 \cdot \frac{kip \cdot ft}{ft}$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI}}{F_{VFtLC4StrI}} = 1.433 \cdot ft$$

Maximum and minimum bearing pressure

$$q_{max} := \frac{F_{VFtLC4StrI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 4.834 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC4StrI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 1.072 \cdot ksf$$

The critical section for flexural design of the toe is at the front face of the wall. **LRFD 5.12.8.4**

Bearing stress at the critical section

$$q_{toe} := q_{min} + \frac{\left(q_{max} - q_{min}\right)}{B_{footing}} \cdot \left(B_{footing} - l_{toe}\right) = 3.069 \cdot ksf$$

A simplified analysis method is used in this example to determine the maximum moments at the front face of the wall by selecting load factors that produce the maximum soil pressure and minimum resisting loads. This method is conservative and eliminates the need for using multiple combinations.

As shown below, minimum load factors are used for the opposing forces, such as the overburden and footing self-weight, to calculate the maximum moment at the front face of the wall.

The moment demand at the critical section:

$$M_{rDemand} := q_{toe} \cdot \frac{l_{toe}^{\,2}}{2} + \left(q_{max} - q_{toe}\right) \cdot \frac{l_{toe}^{\,2}}{3} - 0.9 \cdot W_c \cdot t_{footing} \cdot \frac{l_{toe}^{\,2}}{2} - 1.0 \gamma_s \cdot \left(h_{toeDepth} - t_{footing}\right) \cdot \frac{l_{toe}^{\,2}}{2}$$

$$M_{rDemand} = 67.396 \cdot \frac{kip \cdot ft}{ft}$$

Flexure Resistance

LRFD 5.6.3.2

The design procedure consists of calculating the reinforcing steel area required to satisfy the moment demand, and checking the selected steel area against the requirements and limitations for developing an adequate moment capacity, controlling crack width, and managing shrinkage and temperature stresses.

As a trial, select No. 8 bars. $\quad$ $bar := 8$

Nominal diameter of a reinforcing steel bar $\quad$ $d_{bar} := Dia(bar) = 1 \cdot in$

Cross-section area of a reinforcing steel bar on the flexural tension side $\quad$ $A_{bar} := Area(bar) = 0.79 \cdot in^2$

The spacing shall not exceed 12 in. when the footing thickness is greater than 18 in. **LRFD 5.10.6**

Footing thickness $\quad$ $t_{footing} = 3\,ft$

Selected spacing of reinforcing steel bars $\quad$ $s_{bar} := 12 \cdot in$

Area of tension steel in a 1 ft wide strip $A_{sProvided} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.79 \cdot in^2$

Effective depth $d_e := t_{footing} - Cover_{ft} = 32 \cdot in$

Resistance factor for flexure $\phi_f := 0.9$ **LRFD 5.5.4.2**

A 1-ft wide strip is selected for the design.

Width of the compression face of the section $b := 12in$

Stress block factor $\beta_1 := \min\left[\max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4ksi}{ksi}\right), 0.65\right], 0.85\right] = 0.85$ **LRFD 5.6.2.2**

Solve the following quadratic equation to calculate the required A_s needed to satisfy the moment demand. Use an assumed initial A_s value to solve the quadratic equation.

The initial assumption for A_s $A_s := 1in^2$

Solve the quadratic equation for the area of steel required

$$\text{Given} \quad M_{rDemand} \cdot ft = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$$

Required steel area $A_{sRequired} := Find(A_s) = 0.475 \cdot in^2$

Check if $A_{sProvided} > A_{sRequired}$ $Check := if(A_{sProvided} > A_{sRequired}, "OK", "Not OK") = "OK"$

Moment capacity of the section with the provided steel area

$$M_{Provided} := \phi_f \cdot A_{sProvided} \cdot f_y \cdot \frac{\left[d_e - \frac{1}{2} \cdot \left(\frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]}{ft}$$

$$M_{Provided} = 111.007 \cdot \frac{kip \cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis $c := \frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.82 \cdot in$

Check the validity of the assumption, $f_s = f_y$ $Check\_f_s := if\left(\frac{c}{d_e} < 0.6, "OK", "Not OK"\right) = "OK"$

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must develop a factored flexural resistance equal to the lesser of the cracking moment or 1.33 times the factored moment from the applicable strength limit state load combinations.

Flexural cracking variability factor $\gamma_1 := 1.6$ For concrete structures that are not precast segmental

Ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement $\gamma_3 := 0.67$ For ASTM615 grade 60 reinforcement

Section modulus $S_c := \frac{1}{6} \cdot b \cdot t_{footing}^{2} = 2.592 \times 10^{3} \cdot in^{3}$

Cracking moment $M_{cr} := \frac{\gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c}{ft} = 96.254 \cdot \frac{kip \cdot ft}{ft}$

1.33 times the factored moment demand $1.33 \cdot M_{rDemand} = 89.637 \cdot \frac{kip \cdot ft}{ft}$

Required moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33 M_{rDemand}, M_{cr}\right) = 89.637 \cdot \frac{kip \cdot ft}{ft}$

Check the adequacy of section capacity $Check := if\left(M_{Provided} > M_{req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions extends the service life. The width of potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress.

The spacing requirement for the mild steel reinforcement in the layer closer to the tension face $s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ **LRFD Eq. 5.6.7-1**

Exposure factor for Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest bar $d_c := Cover_{ft} + \frac{d_{bar}}{2} = 4.5 \cdot in$

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face $\beta_s := 1 + \frac{d_c}{0.7\left(t_{footing} - d_c\right)} = 1.204$

The calculation of tensile stress in nonprestressed reinforcement at the service limit state, f_{ss}, requires establishing the neutral axis location and the moment demand at the critical section.

The position of the section's neutral axis is determined through an iterative process to calculate the actual stress in the reinforcement. This process starts with an assumed position of the neutral axis.

Assumed position of the neutral axis $x := 5 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvided} \cdot \left(d_e - x\right)$

Position of the neutral axis $x_{na} := Find(x) = 5.303 \cdot in$

The vertical force and moment at the base of the footing from Load Case IV under Service I limit state are:

$F_{VFtLC4SerI} = 30.702 \cdot \frac{kip}{ft}$ $M_{uFtLC4SerI} = 31.183 \cdot \frac{kip \cdot ft}{ft}$

Eccentricity in the footing width direction under Service I limit state $e_{BSerI} := \frac{M_{uFtLC4SerI}}{F_{VFtLC4SerI}} = 1.016 \cdot ft$

Maximum and minimum soil pressure under Service I limit state

$$q_{maxSerI} := \frac{F_{VFtLC4SerI}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_{BSerI}}{B_{footing}}\right) = 3.301 \cdot ksf$$

$$q_{minSerI} := \frac{F_{VFtLC4SerI}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_{BSerI}}{B_{footing}}\right) = 1.248 \cdot ksf$$

Soil pressure at the critical section under Service I limit state

$$q_{toeSerI} := q_{minSerI} + \frac{\left(q_{maxSerI} - q_{minSerI}\right)}{B_{footing}} \cdot \left(B_{footing} - l_{toe}\right)$$

$$q_{toeSerI} = 2.338 \cdot ksf$$

The moment at the critical section under Service I limit state:

$$M_{rSerI} := q_{toeSerI} \cdot \frac{l_{toe}^{2}}{2} + \left(q_{maxSerI} - q_{toeSerI}\right) \cdot \frac{l_{toe}^{2}}{3} - W_c \cdot t_{footing} \cdot \frac{l_{toe}^{2}}{2} - \gamma_s \cdot \left(h_{toeDepth} - t_{footing}\right) \cdot \frac{l_{toe}^{2}}{2}$$

$$M_{rSerI} = 41.109 \cdot \frac{kip \cdot ft}{ft}$$

Tensile force in the reinforcing steel due to service I limit state moment

$$T_s := \frac{M_{rSerI}}{d_e - \frac{x_{na}}{3}} \cdot ft = 16.3 \cdot kip$$

Stress in the reinforcing steel due to the service I limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided}} = 20.655 \cdot ksi$$

f_{ss} not to exceed $0.6f_y$

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 20.655 \cdot ksi$$

Required reinforcement spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 19.146 \cdot in$$

Check if the spacing provided < the required spacing

$$Check := if\left(s_{bar} < s_{barRequired}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shrinkage and Temperature Reinforcement

LRFD 5.10.6

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the toe

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := \min\left[\left[\begin{array}{c} \left(0.60 \frac{in^2}{ft}\right) \\ \max\left[\left[\begin{array}{c} \left(0.11 \frac{in^2}{ft}\right) \\ \left[\frac{1.3 \cdot B_{footing} \cdot t_{footing} \cdot \frac{kip}{in \cdot ft}}{2\left(B_{footing} + t_{footing}\right) \cdot f_y}\right] \end{array}\right]\right] \end{array}\right]\right] \cdot ft = 0.319 \cdot in^2$$

Check if the provided area of steel > the required area of shrinkage and temperature steel

$$Check := if\left(A_{sProvided} > A_{shrink.temp}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Design for Shear

Effective width of the section $b = 12 \cdot in$

Depth of equivalent rectangular stress block $a := \frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot b} = 1.549 \cdot in$

Effective shear depth $d_v := \max\left(d_e - \frac{a}{2}, 0.9 \cdot d_e, 0.72 \cdot t_{footing}\right) = 31.225 \cdot in$ **LRFD 5.7.2.8**

The critical section for shear at the toe side is located at a distance d_v from the front face of the wall.

Distance from the toe to the shear critical section $l_{shear} := l_{toe} - d_v = 3.731\ ft$

Bearing stress at the shear critical section $q_d := q_{min} + \frac{(q_{max} - q_{min})}{B_{footing}} \cdot (B_{footing} - l_{shear}) = 3.794 \cdot ksf$

Minimum load factors are used for the opposing forces such as the overburden and footing self weight to calculate the maximum shear at the shear critical section.

Factored shear demand at the shear critical section

$$V_{uFtToe} := \frac{(q_{max} + q_d)}{2} \cdot l_{shear} - 0.9 \cdot W_c \cdot t_{footing} \cdot l_{shear} - 1.0 \cdot \gamma_s \cdot (h_{toeDepth} - t_{footing}) \cdot l_{shear}$$

$$V_{uFtToe} = 12.794 \cdot \frac{kip}{ft}$$

For concrete footing in which the distance from point of zero shear to the face of the wall is less than $3d_v$, the simplified procedure for nonprestressed sections can be used. **LRFD 5.7.3.4.1**

Check if the distance l_{toe} is less than $3d_v$ $Check := if(l_{toe} < 3 \cdot d_v, "Yes", "No") = "Yes"$

Therefore, the simplified procedure is used.

Factor indicating the ability of diagonally cracked concrete to transmit tension and shear $\beta := 2$

Nominal shear resistance of concrete, V_n, is calculated as follows:

$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_e = 42 \cdot kip$ **LRFD Eq. 5.7.3.3-3**

$V_{c2} := 0.25 f_c \cdot b \cdot d_e = 288 \cdot kip$ **LRFD Eq. 5.7.3.3-2**

$V_n := \min(V_{c1}, V_{c2}) = 42.035 \cdot kip$

Resistance factor for shear $\phi_v := 0.9$ **LRFD 5.5.4.2**

Factored shear resistance $V_r := \phi_v \cdot V_n = 37.831 \cdot kip$

Check if the factored shear resistance is greater than the shear demand $Check := if\left(\frac{V_r}{ft} > V_{uFtToe}, "OK", "Not OK"\right) = "OK"$

Development Length of Reinforcement

The flexural reinforcing steel must be developed on each side of the critical section for its full development length. **LRFD 5.10.8.1.2**

Available development length $l_{d.available} := l_{toe} - Cover_{ft} = 6\,ft$

Basic development length $l_{db} := 2.4 \cdot d_{bar} \cdot \dfrac{f_y}{\sqrt{f_c \cdot ksi}} = 6.928\,ft$ **LRFD Eq. 5.10.8.2.1a-2**

Reinforcement location factor $\lambda_{rl} := 1$ No more than 12 in. concrete below

Coating factor $\lambda_{cf} := 1.5$ Epoxy coated bars with less than $3d_b$ cover

Reinforcement confinement factor $\lambda_{rc} := 0.4$ For $c_b > 2.5$ in. and No. 8 bars or smaller

Excess reinforcement factor $\lambda_{er} := \dfrac{A_{sRequired}}{A_{sProvided}} = 0.601$ **LRFD Eq. 5.10.8.2.1c-4**

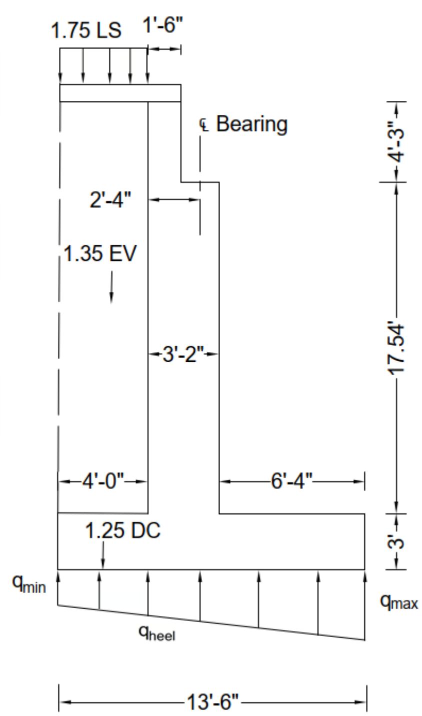
Normal weight concrete $\lambda := 1$

Required bar development length $l_{d.required} := l_{db} \cdot \dfrac{(\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = 2.499\,ft$ **LRFD Eq. 5.10.8.2.1a-1**

Check if $l_{d.available} > l_{d.required}$ $Check := if(l_{d.available} > l_{d.required}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$

Heel Design

A summary of dimensions and loads for the design is shown in the following figure.



The loads that act directly on the heel are the self-weight of the footing, soil weight, live load surcharge and the resisting soil pressure from the bottom of the footing. The critical load combination for the heel design uses the load factors producing the minimum axial loads and maximum eccentricities, which results in the minimum soil pressure at the bottom of the footing.

The critical location for the flexural design is at the back face of the wall. **LRFD 5.12.8.4**

In the general case of a cantilever abutment wall, where the downward load on the heel is larger than the upward reaction of the soil under the heel, the top of the heel is in tension. Therefore, the critical section for shear is taken at the back face of the abutment wall. **LRFD C5.12.8.6.1**

Load cases I, III, and IV under Strength I limit state are considered in calculating the maximum moment and shear at the critical section.

Load Case I

Minimum vertical force

$$F_{VFtLC1StrIMin} = 21.662 \cdot \frac{kip}{ft}$$

Step 3.6, sliding resistance check

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC1StrI} = 38.136 \cdot \frac{kip \cdot ft}{ft}$$

Step 3.6, eccentric load limitation check

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC1StrI}}{F_{VFtLC1StrIMin}} = 1.761 \cdot ft$$

Maximum and minimum soil pressure

$$q_{max} := \frac{F_{VFtLC1StrIMin}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 2.86 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC1StrIMin}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 0.349 \cdot ksf$$

Bearing stress at the critical section

$$q_{heelLC1StrI} := q_{min} + \left(q_{max} - q_{min}\right) \frac{l_{heel}}{B_{footing}} = 1.093 \cdot ksf$$

Factored moment at the critical section

$$M_{rLC1StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot \frac{{l_{heel}}^2}{2} + 1.35EV_{earthBk} \cdot \frac{l_{heel}}{2} - q_{min} \cdot l_{heel} \cdot \frac{l_{heel}}{2} - \frac{1}{6}\left(q_{heelLC1StrI} - q_{min}\right) {l_{heel}}^2$$

$$M_{rLC1StrI} = 12.671 \cdot \frac{kip \cdot ft}{ft}$$

Factored shear force at the critical section

$$V_{uHeelLC1StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35EV_{earthBk} - q_{min} \cdot l_{heel} - \frac{1}{2} \cdot \left(q_{heelLC1StrI} - q_{min}\right) \cdot l_{heel}$$

$$V_{uHeelLC1StrI} = 5.839 \cdot \frac{kip}{ft}$$

Load Case III

There are two cases that need to be considered: without and with live load.

Without live load:

Minimum vertical force

$$F_{VFtLC3StrIMin\_noLL} = 26.754 \cdot \frac{kip}{ft}$$

Step 3.6, sliding resistance check

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3StrI\_noLL} = 34.635\ ft \cdot \frac{kip}{ft}$$

Step 3.6, eccentric load limitation check

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC3StrI\_noLL}}{F_{VFtLC3StrIMin\_noLL}} = 1.295 \cdot ft$$

Maximum and minimum soil pressures

$$q_{max} := \frac{F_{VFtLC3StrIMin\_noLL}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.122 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC3StrIMin\_noLL}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 0.842 \cdot ksf$$

Bearing stress at the critical section

$$q_{heelLC3StrI} := q_{min} + \left(q_{max} - q_{min}\right)\frac{l_{heel}}{B_{footing}} = 1.517 \cdot ksf$$

Factored moment at the critical section

$$M_{rLC3StrI\_noLL} := 1.25 \cdot W_c \cdot t_{footing} \cdot \frac{l_{heel}^2}{2} + 1.35 EV_{earthBk} \cdot \frac{l_{heel}}{2} - q_{min} \cdot l_{heel} \cdot \frac{l_{heel}}{2} - \frac{1}{6}\left(q_{heelLC3StrI} - q_{min}\right) l_{heel}^2$$

$$M_{rLC3StrI\_noLL} = 8.913 \cdot \frac{kip \cdot ft}{ft}$$

Factored shear force at the critical section

$$V_{uHeelLC3StrI\_noLL} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35 EV_{earthBk} - q_{min} \cdot l_{heel} - \frac{1}{2} \cdot \left(q_{heelLC3StrI} - q_{min}\right) \cdot l_{heel}$$

$$V_{uHeelLC3StrI\_noLL} = 4.006 \cdot \frac{kip}{ft}$$

With live load:

Minimum vertical force

$$F_{VFtLC3StrIMin} = 36.304 \cdot \frac{kip}{ft}$$

Step 3.6, sliding resistance check

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3StrI} = 30.656 \cdot \frac{kip \cdot ft}{ft}$$

Step 3.6, summary table

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC3StrI}}{F_{VFtLC3StrIMin}} = 0.844 \cdot ft$$

Maximum and minimum soil pressure

$$q_{max} := \frac{F_{VFtLC3StrIMin}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.698 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC3StrIMin}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 1.68 \cdot ksf$$

Bearing stress at the critical section

$$q_{heelLC3StrI} := q_{min} + \left(q_{max} - q_{min}\right) \frac{l_{heel}}{B_{footing}} = 2.278 \cdot ksf$$

Factored moment at the critical section

$$M_{rLC3StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot \frac{l_{heel}^{\,2}}{2} + 1.35EV_{earthBk} \cdot \frac{l_{heel}}{2} - q_{min} \cdot l_{heel} \cdot \frac{l_{heel}}{2} - \frac{1}{6}\left(q_{heelLC3StrI} - q_{min}\right) l_{heel}^{\,2}$$

$$M_{rLC3StrI} = 2.413 \cdot \frac{kip \cdot ft}{ft}$$

Factored shear force at the critical section

$$V_{uHeelLC3StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35EV_{earthBk} - q_{min} \cdot l_{heel} - \frac{1}{2} \cdot \left(q_{heelLC3StrI} - q_{min}\right) \cdot l_{heel}$$

$$V_{uHeelLC3StrI} = 0.808 \cdot \frac{kip}{ft}$$

Load Case IV

There are two cases that need to be considered: without and with live load surcharge.

Without live load surcharge:

Minimum vertical force

$$F_{VFtLC4StrIMin\_noLS} = 26.754 \cdot \frac{kip}{ft}$$

Step 3.6, sliding resistance check

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4StrI\_noLS} = 38.891 \cdot \frac{kip \cdot ft}{ft}$$

Step 3.6, eccentric load limitation check

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI\_noLS}}{F_{VFtLC4StrIMin}} = 1.368 \cdot ft$$

Maximum and minimum soil pressures

$$q_{max} := \frac{F_{VFtLC4StrIMin\_noLS}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.186 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC4StrIMin\_noLS}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 0.777 \cdot ksf$$

Bearing stress at the critical section

$$q_{heelLC4StrI} := q_{min} + \left(q_{max} - q_{min}\right) \frac{l_{heel}}{B_{footing}} = 1.491 \cdot ksf$$

Factored moment at the critical section

$$M_{rLC4StrI\_noLS} := 1.25 \cdot W_c \cdot t_{footing} \cdot \frac{l_{heel}^{2}}{2} \ldots$$
$$+ 1.35 EV_{earthBk} \cdot \frac{l_{heel}}{2} - q_{min} \cdot l_{heel} \cdot \frac{l_{heel}}{2} - \frac{1}{6}\left(q_{heelLC4StrI} - q_{min}\right) l_{heel}^{2}$$

$$M_{rLC4StrI\_noLS} = 9.327 \cdot \frac{kip \cdot ft}{ft}$$

Factored shear force at the critical section

$$V_{uHeelLC4StrI\_noLS} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35 EV_{earthBk} - q_{min} \cdot l_{heel} - \frac{1}{2} \cdot \left(q_{heelLC4StrI} - q_{min}\right) \cdot l_{heel}$$

$$V_{uHeelLC4StrI\_noLS} = 4.187 \cdot \frac{kip}{ft}$$

With live load surcharge:

Minimum vertical force

$$F_{VFtLC4StrIMin} = 28.434 \cdot \frac{kip}{ft}$$

Step 3.6, sliding resistance check

Step 3.6, summary table

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4StrI} = 57.133 \cdot \frac{kip \cdot ft}{ft}$$

Eccentricity in the footing width direction

$$e_B := \frac{M_{uFtLC4StrI}}{F_{VFtLC4StrIMin}} = 2.009 \cdot ft$$

Maximum and minimum soil pressure

$$q_{max} := \frac{F_{VFtLC4StrIMin}}{B_{footing}} \cdot \left(1 + \frac{6 \cdot e_B}{B_{footing}}\right) = 3.987 \cdot ksf$$

$$q_{min} := \frac{F_{VFtLC4StrIMin}}{B_{footing}} \cdot \left(1 - \frac{6 \cdot e_B}{B_{footing}}\right) = 0.225 \cdot ksf$$

Bearing stress at the critical section

$$q_{heelLC4StrI} := q_{min} + \left(q_{max} - q_{min}\right) \frac{l_{heel}}{B_{footing}} = 1.34 \cdot ksf$$

Factored moment at the critical section

$$M_{rLC4StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot \frac{l_{heel}^{2}}{2} + 1.35 EV_{earthBk} \cdot \frac{l_{heel}}{2} - q_{min} \cdot l_{heel} \cdot \frac{l_{heel}}{2} - \frac{1}{6}\left(q_{heelLC4StrI} - q_{min}\right) l_{heel}^{2}$$

$$M_{rLC4StrI} = 12.672 \cdot \frac{kip \cdot ft}{ft}$$

Factored shear force at the critical section

$$V_{uHeelLC4StrI} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35 EV_{earthBk} - q_{min} \cdot l_{heel} - \frac{1}{2} \cdot \left(q_{heelLC4StrI} - q_{min}\right) \cdot l_{heel}$$

$$V_{uHeelLC4StrI} = 5.593 \cdot \frac{kip}{ft}$$

Moment demand at the critical section

$$M_{HeelDemand} := \max\left(M_{rLC1StrI}, M_{rLC3StrI\_noLL}, M_{rLC3StrI}, M_{rLC4StrI\_noLS}, M_{rLC4StrI}\right) = 12.672 \cdot \frac{kip \cdot ft}{ft}$$

Shear demand at the critical section

$$V_{HeelDemand} := \max\left(V_{uHeelLC1StrI}, V_{uHeelLC3StrI\_noLL}, V_{uHeelLC3StrI}, V_{uHeelLC4StrI\_noLS}, V_{uHeelLC4StrI}\right)$$

$$V_{HeelDemand} = 5.839 \cdot \frac{kip}{ft}$$

Flexure Resistance

LRFD 5.6.3.2

The design procedure consists of calculating the reinforcing steel area required to satisfy the moment demand, and checking the selected steel area against the requirements and limitations for developing an adequate moment capacity, controlling crack width, and managing shrinkage and temperature stresses.

As a trial, select No. 6 bars. $bar := 6$

Nominal diameter of a reinforcing steel bar $d_{bar} := Dia(bar) = 0.75 \cdot in$

Cross-section area of a reinforcing steel bar on the flexural tension side $A_{bar} := Area(bar) = 0.44 \cdot in^2$

The spacing shall not exceed 12 in. when the footing thickness is greater than 18 in. **LRFD 5.10.6**

Footing thickness $t_{footing} = 3\,ft$

Selected spacing of reinforcing bars $s_{bar} := 12 \cdot in$

Area of tension steel in a 1 ft wide strip $A_{sProvided} := \dfrac{A_{bar} \cdot 12in}{s_{bar}} = 0.44 \cdot in^2$

Effective depth $d_e := t_{footing} - Cover_{ft} = 32 \cdot in$

Resistance factor for flexure $\phi_f := 0.9$ **LRFD 5.5.4.2**

A 1-ft wide strip is selected for the design.

Width of the compression face of the section $b := 12in$

Stress block factor $\beta_1 = 0.85$

Solve the following quadratic equation to calculate the required A_s needed to satisfy the moment demand. Use an assumed initial A_s value to solve the quadratic equation.

Initial assumption $A_s := 1in^2$

Solve the quadratic equation for the area of steel required

$$\text{Given} \quad M_{HeelDemand} \cdot ft = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$$

Required steel area $A_{sRequired} := Find(A_s) = 0.088 \cdot in^2$

Check if $A_{sProvided} > A_{sRequired}$ $Check := if\left(A_{sProvided} > A_{sRequired}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Moment capacity of the section with the provided steel area

$$M_{Provided} := \phi_f \cdot A_{sProvided} \cdot f_y \cdot \frac{\left[d_e - \frac{1}{2}\cdot\left(\frac{A_{sProvided}\cdot f_y}{0.85\cdot f_c\cdot b}\right)\right]}{ft}$$

$$M_{Provided} = 62.506 \cdot \frac{kip\cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis

$$c := \frac{A_{sProvided}\cdot f_y}{0.85\cdot f_c\cdot \beta_1\cdot b} = 1.01\cdot in$$

Check the validity of the assumption, $f_s = f_y$

$$Check\_f_s := \text{if}\left(\frac{c}{d_e} < 0.6, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must develop a factored flexural resistance equal to the lesser of the cracking moment or 1.33 times the factored moment from the applicable strength limit state load combinations.

Flexural cracking variability factor $\gamma_1 := 1.6$ For concrete structures that are not precast segmental

Ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement $\gamma_3 := 0.67$ For ASTM615 grade 60 reinforcement

Section modulus

$$S_c := \frac{1}{6}\cdot b\cdot t_{footing}^{\ 2} = 2.592\times 10^3\cdot in^3$$

Cracking moment

$$M_{cr} := \frac{\gamma_3\cdot\gamma_1\cdot f_r\cdot S_c}{ft} = 96.254\cdot\frac{kip\cdot ft}{ft}$$

1.33 times the factored moment demand

$$1.33\cdot M_{HeelDemand} = 16.854\cdot\frac{kip\cdot ft}{ft}$$

Required moment to satisfy the minimum reinforcement requirement

$$M_{req} := \min\left(1.33M_{HeelDemand}, M_{cr}\right) = 16.854\cdot\frac{kip\cdot ft}{ft}$$

Check the adequacy of the section capacity

$$Check := \text{if}\left(M_{Provided} > M_{req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions extends the service life. The width of potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress.

The spacing requirement of the mild steel reinforcement in the layer closer to the tension face

$$s \le \frac{700\cdot\gamma_e}{\beta_s\cdot f_{ss}} - 2\cdot d_c$$

LRFD Eq. 5.6.7-1

Exposure factor for Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest bar

$$d_c := Cover_{ft} = 4 \cdot in$$

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer closest to the tension face

$$\beta_s := 1 + \frac{d_c}{0.7\left(t_{footing} - d_c\right)} = 1.179$$

The calculation of tensile stress in nonprestressed reinforcement at the service limit state, f_{ss}, requires establishing the neutral axis location and the moment demand at the critical section.

The position of the section's neutral axis is determined through an iterative process to calculate the actual stress in the reinforcement. This process starts with an assumed position of the neutral axis.

Assumed position of the neutral axis

$$x := 5 \cdot in$$

Given

$$\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvided} \cdot \left(d_e - x\right)$$

Position of the neutral axis

$$x_{na} := Find(x) = 4.049 \cdot in$$

Maximum and minimum soil pressure under Service I limit state, calculated for the toe design

$$q_{maxSerI} = 3.301 \cdot ksf \qquad q_{minSerI} = 1.248 \cdot ksf$$

Soil pressure at the critical section

$$q_{HeelSerI} := q_{minSerI} + \frac{\left(q_{maxSerI} - q_{minSerI}\right)}{B_{footing}} \cdot l_{heel} = 1.856 \cdot ksf$$

The moment at the critical section under Service I limit state:

$$M_{heelSerI} := W_c \cdot t_{footing} \cdot \frac{l_{heel}^{\;2}}{2} + EV_{earthBk} \cdot \frac{l_{heel}}{2} \; ...$$

$$+ V_{LSFooting} \cdot \frac{l_{heel}}{2} - q_{minSerI} \cdot \frac{l_{heel}^{\;2}}{2} - \left(q_{HeelSerI} - q_{minSerI}\right) \cdot \frac{l_{heel}^{\;2}}{6}$$

$$M_{heelSerI} = 3.507 \cdot \frac{kip \cdot ft}{ft}$$

Tensile force in the reinforcing steel due to service limit state moment

$$T_s := \frac{M_{heelSerI}}{d_e - \frac{x_{na}}{3}} \cdot ft = 1.4 \cdot kip$$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided}} = 3.12 \cdot ksi$$

f_{ss} not to exceed $0.6f_y$

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 3.12 \cdot ksi$$

Required reinforcement spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 182.337 \cdot in$$

Check if the spacing provided < the required spacing

$$Check := if\left(s_{bar} < s_{barRequired}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shrinkage and Temperature Reinforcement

LRFD 5.10.6

The required minimum shrinkage and temperature reinforcement area was calculated previously for the toe.

Required shrinkage and temperature steel area $\quad A_{shrink.temp} = 0.319 \cdot in^2$

Check if the provided area of steel > the required area of shrinkage and temperature steel $\quad Check := if(A_{sProvided} > A_{shrink.temp}, "OK", "Not OK") = "OK"$

Design for Shear

The critical section for shear in the heel is at the back face of the abutment wall.

Shear demand at the critical section $\quad V_{HeelDemand} = 5.839 \cdot \frac{kip}{ft}$

Effective width of the section $\quad b = 12 \cdot in$

Depth of an equivalent rectangular stress block $\quad a := \frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.863 \cdot in$

Effective shear depth $\quad d_v := \max\left(d_e - \frac{a}{2}, 0.9 \cdot d_e, 0.72 \cdot t_{footing}\right) = 31.569 \cdot in$ **LRFD 5.7.2.8**

For a concrete footing, in which the distance from point of zero shear to the face of the wall is less than $3d_v$, the simplified procedure for nonprestressed sections can be used. **LRFD 5.7.3.4.1**

Check if the distance l_{heel} is less than $3d_v$ $\quad Check := if(l_{heel} < 3 \cdot d_v, "Yes", "No") = "Yes"$

Therefore, the simplified procedure can be used.

Factor indicating the ability of diagonally cracked concrete to transmit tension and shear $\quad \beta := 2$

Nominal shear resistance of concrete, V_n, is calculated as follows:

$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_e = 42 \cdot kip$ **LRFD Eq. 5.7.3.3-3**

$V_{c2} := 0.25 f_c \cdot b \cdot d_e = 288 \cdot kip$ **LRFD Eq. 5.7.3.3-2**

$V_n := \min(V_{c1}, V_{c2}) = 42.035 \cdot kip$

Resistance factor for shear $\quad \phi_v := 0.9$ **LRFD 5.5.4.2**

Factored shear resistance $\quad V_r := \phi_v \cdot V_n = 37.831 \cdot kip$

Check if the factored shear resistance is greater than the shear demand $\quad Check := if\left(\frac{V_r}{ft} > V_{HeelDemand}, "OK", "Not OK"\right) = "OK"$

Development Length of Reinforcement

The flexural reinforcing steel must be developed on each side of the critical section for its full development length. **LRFD 5.10.8.1.2**

Available development length $l_{d.available} := l_{heel} - Cover_{ft} = 44 \cdot in$

Basic development length $l_{db} := 2.4 \cdot d_{bar} \cdot \frac{f_y}{\sqrt{f_c \cdot ksi}} = 5.196\ ft$ **LRFD Eq. 5.10.8.2.1a-2**

Reinforcement location factor $\lambda_{rl} := 1.3$ More than 12 in. concrete below

Coating factor $\lambda_{cf} := 1.5$ Epoxy coated bars with less than $3d_b$ cover

Reinforcement confinement factor $\lambda_{rc} := 0.4$ For $c_b > 2.5$ in. and No. 8 bars or smaller

Excess reinforcement factor $\lambda_{er} := \frac{A_{sRequired}}{A_{sProvided}} = 0.201$ **LRFD Eq. 5.10.8.2.1c-4**

Normal weight concrete $\lambda := 1$

Required development length $l_{d.required} := l_{db} \cdot \frac{(\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = 9.754 \cdot in$ **LRFD Eq. 5.10.8.2.1a-1**

Check if $l_{d.available} > l_{d.required}$ $Check := if(l_{d.available} > l_{d.required}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$

Shrinkage and Temperature Reinforcement Design

The following calculations check the required amount of reinforcing steel in the secondary direction to control shrinkage and temperature stresses in the footing.

The reinforcement along the longitudinal direction of the footing at the top and bottom should satisfy the shrinkage and temperature reinforcement requirement. **LRFD 5.10.6**

The spacing of reinforcement shall not exceed 12 in. when the footing thickness is greater than 18 in. **LRFD 5.10.6**

MDOT practice is to use 18 in. as the maximum spacing. **BDG 5.16.01**

As a trial, select No. 6 bars. $bar := 6$

Nominal diameter of a reinforcing bar $d_{bST} := Dia(bar) = 0.75 \cdot in$

Cross-section area of the bar $A_{barST} := Area(bar) = 0.44 \cdot in^2$

Selected bar spacing $s_{barST} := 12 \cdot in$

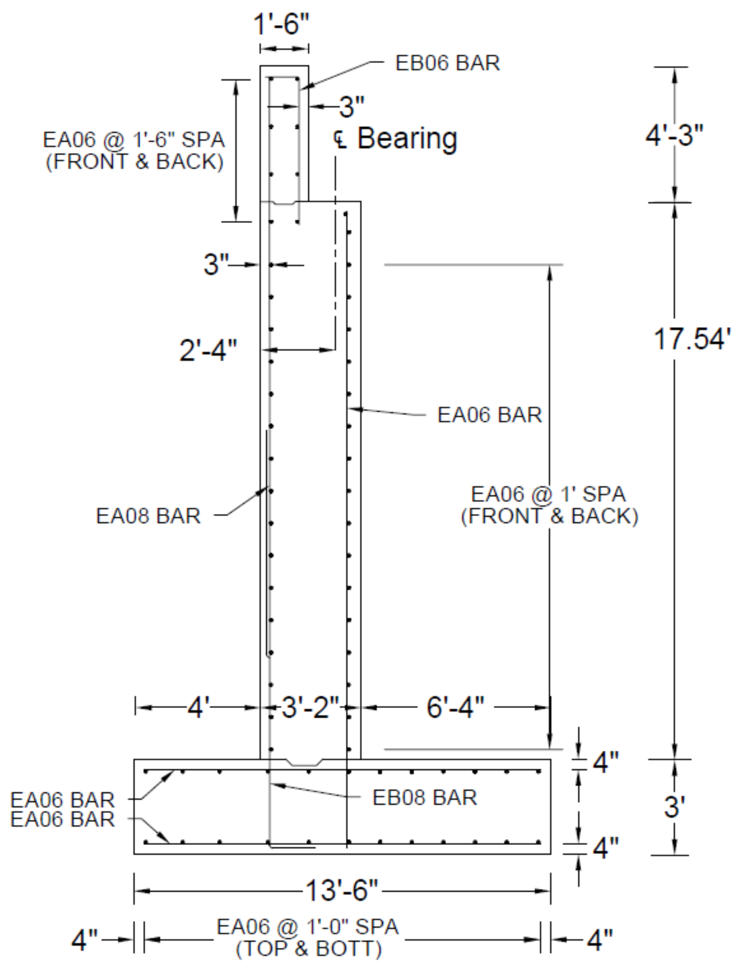
Provided horizontal reinforcement area $A_{sProvidedST} := \frac{A_{barST} \cdot 12in}{s_{barST}} = 0.44 \cdot in^2$

Required minimum area of shrinkage and temperature reinforcement in the footing $A_{shrink.temp} = 0.319 \cdot in^2$

Check if the provided steel area > the required shrinkage and temperature steel area $Check := if(A_{sProvidedST} > A_{shrink.temp}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$

The footing design presented in this step provides the following details:

- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the transverse flexural reinforcement at the top of the footing.
- No. 8 bars at 12.0 in. spacing ($A_s = 0.79$ in.^{2}/ft) as the transverse flexural reinforcement at the bottom of the footing.
- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the longitudinal shrinkage and temperature reinforcement at the top and bottom of the footing.



Note: Refer to MDOT BDG for additional bars, laps, embedment, and keyway dimensions. They are not shown in this drawing for clarity of main reinforcement.

References

MDOT Geotechnical Manual (2019).
https://www.michigan.gov/-/media/Project/Websites/MDOT/Programs/Bridges-and-Structures/Geotechnical-Services/Geotechnical-Manual.pdf?rev=00901c15702e4493963ee866d0ed4c01 (Last accessed: 09/30/2022)

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Section 4 Abutment with Piles

Step 4.1 Preliminary Abutment Dimensions

Description

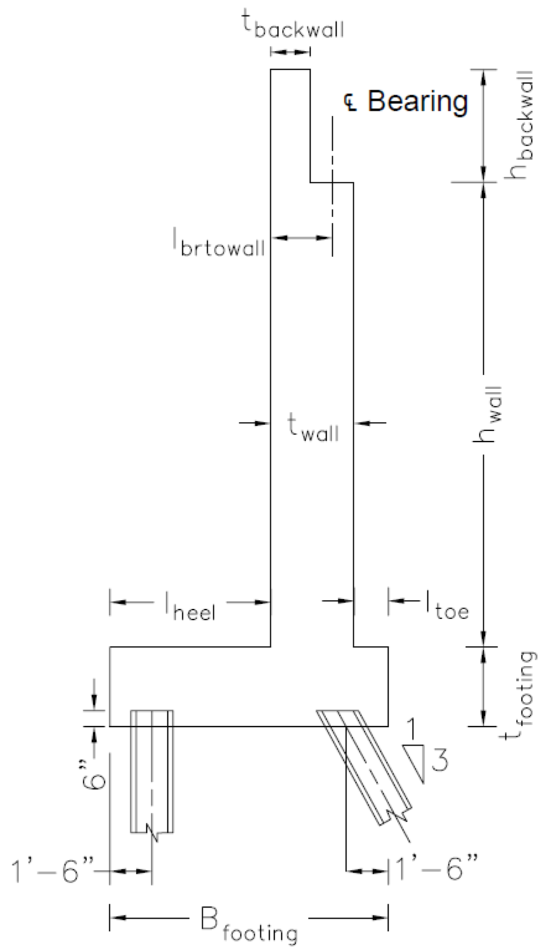
This step presents the selected preliminary abutment dimensions.

The design criteria, bridge information, material properties, reinforcing steel cover requirements, soil types and properties, and superstructure loads are taken from Section 2.

This section presents the design of a full-depth reinforced concrete cantilever abutment with pile supports.

The structural design of backwall and abutment wall is presented in Section 2. The pile design presented in this example focuses on structural design assuming that the geotechnical design is performed by a geotechnical engineer.

The designer should select the preliminary dimensions based on state specific standards, previous designs, and past experience. The following figure shows the abutment geometry and dimensional variables:



The selected preliminary dimensions are listed below:

Abutment length $L_{abut} := W_{deck} = 63.75\ ft$

The abutment has an independent backwall with a sliding deck. **BDG 6.20.03A**

Backwall height $h_{backwall} := 4.25ft$

Backwall thickness $t_{backwall} := 1ft + 6in = 1.5\ ft$

Abutment wall height $h_{wall} := 17.54ft$

Abutment wall thickness $t_{wall} := 3ft + 2in = 3.167\ ft$

Distance from the toe to the front face of the abutment wall $l_{toe} := 1ft + 3in = 1.25\ ft$

Distance from the heel to the back face of the abutment wall $l_{heel} := 6ft + 7in = 6.583\ ft$

Distance from center of the bearing pad to the back face of the abutment wall $l_{brtowall} := 2ft + 4in = 2.333\ ft$

Footing width $B_{footing} := l_{toe} + l_{heel} + t_{wall} = 11\ ft$

Footing length $L_{footing} := L_{abut} + 1ft + 1ft = 65.75\ ft$

Note: The footing extends 1 ft beyond the end of the wall on either side.

Footing thickness $t_{footing} := 3ft$

Toe fill depth to the bottom of the footing $h_{toeDepth} := 7ft$

Step 4.2 Application of Dead Load

Description

This step describes the application of the dead load on the abutment.

The common practice is to apply superstructure dead load as a uniformly distributed load over the length of the abutment. This is accomplished by adding exterior and interior girder end dead load reactions and dividing this quantity by the abutment length.

Dead load of superstructure

Weight of structural components and non-structural attachments (DC)

$$DC_{Sup} := \frac{2 \cdot R_{DCEx} + \left(N_{beams} - 2\right) \cdot R_{DCIn}}{L_{abut}} = 5.658 \cdot \frac{kip}{ft}$$

Weight of the future wearing surface (DW)

$$DW_{Sup} := \frac{2 \cdot R_{DWEx} + \left(N_{beams} - 2\right) \cdot R_{DWIn}}{L_{abut}} = 0.886 \cdot \frac{kip}{ft}$$

Backwall weight

$$DC_{backwall} := h_{backwall} \cdot t_{backwall} \cdot W_c = 0.956 \cdot \frac{kip}{ft}$$

Abutment wall weight

$$DC_{wall} := h_{wall} \cdot t_{wall} \cdot W_c = 8.332 \cdot \frac{kip}{ft}$$

Footing weight

$$DC_{footing} := B_{footing} \cdot t_{footing} \cdot W_c = 4.95 \cdot \frac{kip}{ft}$$

Step 4.3 Application of Live Load

Description

Please refer to Step 2.3 for the application of live load on the structure.

Step 4.4 Application of Other Loads

Description

This step typically includes the calculation of braking force, wind load, earth load, and temperature load.

Since piles and a pile cap are selected to replace the spread footing in Section 2, all other dimensions of the abutment remain consistent. Only the calculation of the earth load is different from Step 2.4. Therefore, please refer to Step 2.4 for the rest of the calculations.

Earth Load

The earth load includes lateral earth pressure, live load surcharge, and vertical earth pressure on the footing. As per the geotechnical engineer, the groundwater table is not located in the vicinity of the foundation. Therefore, the effect of hydrostatic pressure is excluded. The hydrostatic pressure should be avoided, if possible, at abutments and retaining walls through the design of an appropriate drainage system.

Lateral Load Due to Lateral Earth Pressure

The lateral loads due to earth pressure are calculated.

Backwall

Lateral earth pressure at the base $p_{bw} := k_a \cdot \gamma_s \cdot h_{backwall} = 0.153 \cdot ksf$ **LRFD Eq. 3.11.5.1-1**

Lateral load $P_{EHBackwall} := \frac{1}{2} \cdot p_{bw} \cdot h_{backwall} = 0.325 \cdot \frac{kip}{ft}$

Abutment Wall

Lateral earth pressure at the base $p_{wall} := k_a \cdot \gamma_s \cdot (h_{backwall} + h_{wall}) = 0.784 \cdot ksf$

Lateral load $P_{EHWall} := \frac{1}{2} \cdot p_{wall} \cdot (h_{backwall} + h_{wall}) = 8.546 \cdot \frac{kip}{ft}$

Footing

Lateral earth pressure at the base $p_{ft} := k_a \cdot \gamma_s \cdot (h_{backwall} + h_{wall} + t_{footing}) = 0.892 \cdot ksf$

Lateral load $P_{EHFooting} := \frac{1}{2} \cdot p_{ft} \cdot (h_{backwall} + h_{wall} + t_{footing}) = 11.062 \cdot \frac{kip}{ft}$

Vertical Earth Load on the Footing

Back side (heel) $EV_{earthBk} := \gamma_s \cdot l_{heel} \cdot (h_{backwall} + h_{wall}) = 17.214 \cdot \frac{kip}{ft}$

Front side (toe) $EV_{earthFt} := \gamma_s \cdot l_{toe} \cdot (h_{toeDepth} - t_{footing}) = 0.6 \cdot \frac{kip}{ft}$

Live Load Surcharge

Live load surcharge is applied to account for a vehicular live load acting on the backfill surface within a distance equal to one-half the wall height behind the back face of the wall. **LRFD 3.11.6.4**

Height of the abutment $h_{backwall} + h_{wall} + t_{footing} = 24.79\,ft$

Equivalent height of soil for vehicular load $h_{eq} := 2ft$ **LRFD Table 3.11.6.4-1**

Lateral surcharge pressure $\sigma_p := k_a \cdot \gamma_s \cdot h_{eq} = 0.072 \cdot ksf$ **LRFD Eq. 3.11.6.4-1**

Backwall

Lateral load $P_{LSBackwall} := \sigma_p \cdot h_{backwall} = 0.306 \cdot \frac{kip}{ft}$

Abutment Wall

Lateral load $P_{LSWall} := \sigma_p \cdot (h_{backwall} + h_{wall}) = 1.569 \cdot \frac{kip}{ft}$

Footing

Lateral load $P_{LSFooting} := \sigma_p \cdot (h_{backwall} + h_{wall} + t_{footing}) = 1.785 \cdot \frac{kip}{ft}$

Vertical load $V_{LSFooting} := \gamma_s \cdot l_{heel} \cdot h_{eq} = 1.58 \cdot \frac{kip}{ft}$

Step 4.5 Combined Load Effects

Description

This step describes the procedure for combining all load effects and calculating total factored forces and moments acting at the base of the backwall, abutment wall, and footing.

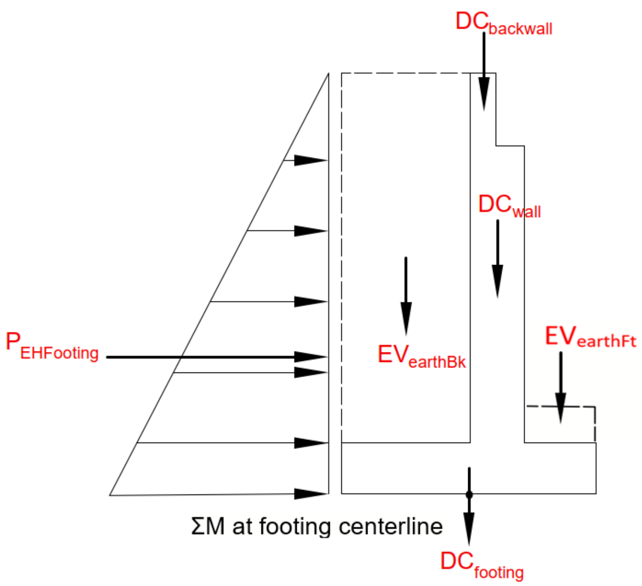
The total factored forces and moments at the base of the backwall and abutment wall are similar to the ones in Step 2.5. Therefore, this step only shows the calculation of the total factored forces and moments at the base of the footing.

Forces and Moments at the Base of the Footing

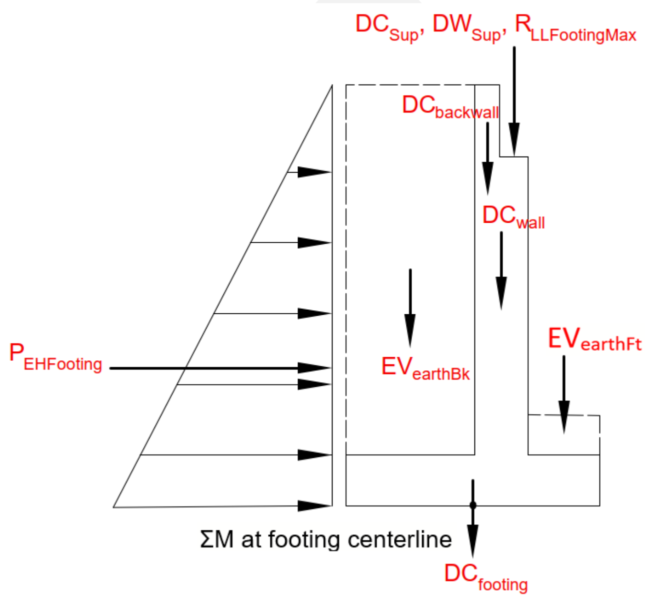
Load Cases I, III, and IV are considered. In addition to all the loads considered for the abutment wall, weight of soil (earth load) on the toe and heel and live load surcharge on the heel are considered.

LRFD 3.6.2.1

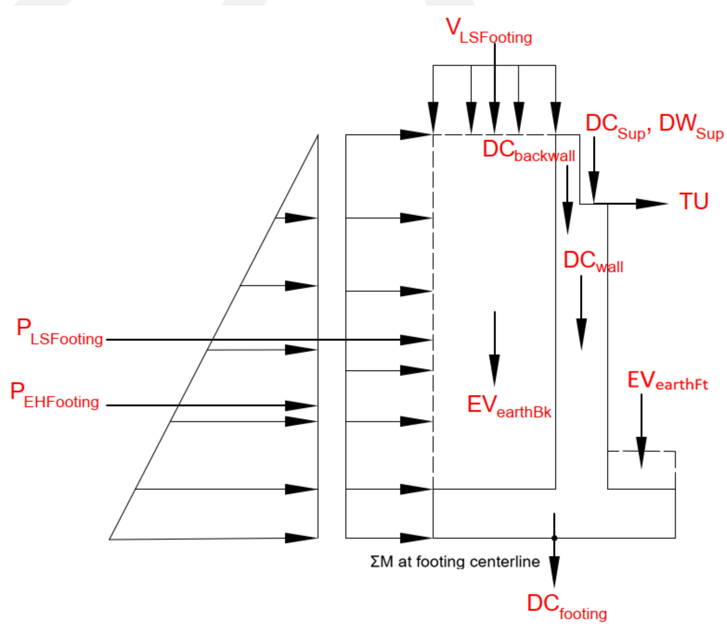
The dynamic load allowance is excluded from the live load for foundation components that are located entirely below the ground level.



LC I



LC III



LC IV

Strength I

Strength I = 1.25DC + 1.5DW + 1.75LL + 1.75BR + 1.5EH + 1.35EV + 1.75LS + 0.5TU

Load Case I

Factored vertical force at the base of the footing

$$F_{VFtLC1StrI} := 1.25\cdot\left(DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.35\cdot\left(EV_{earthBk} + EV_{earthFt}\right) = 41.846\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC1StrI} := 1.5\cdot P_{EHFooting} = 16.593\cdot\frac{kip}{ft}$$

The vertical earth load of the backfill soil reduces the critical moment about the footing longitudinal axis. This requires using the minimum load factor of 1.0 for EV instead of the factor 1.35 in the Strength I combination.

LRFD 3.4.1

This is the same for the moment calculated about the longitudinal axis of the footing for all the load cases and limit states.

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC1StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \dots$$
$$+ 1.5\cdot P_{EHFooting}\cdot\frac{\left(h_{backwall} + h_{wall} + t_{footing}\right)}{3} + 1.35EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right) \dots$$
$$+ 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right)$$

$$M_{uFtLC1StrI} = 133.008\cdot\frac{kip\cdot ft}{ft}$$

Load Case III

Factored vertical force at the base of the footing

$$F_{VFtLC3StrI} := 1.25\cdot\left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.5DW_{Sup} + 1.75R_{LLFootingMax} \dots$$
$$+ 1.35\cdot\left(EV_{earthBk} + EV_{earthFt}\right)$$

$$F_{VFtLC3StrI} = 59.798\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC3StrI} := 1.5\cdot P_{EHFooting} = 16.593\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \dots$$
$$+ \left(1.25\cdot DC_{Sup} + 1.5\cdot DW_{Sup} + 1.75\cdot R_{LLFootingMax}\right)\cdot\left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \dots$$
$$+ 1.5\cdot P_{EHFooting}\cdot\frac{\left(h_{backwall} + h_{wall} + t_{footing}\right)}{3} \dots$$
$$+ 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + 1.35\cdot EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC3StrI} = 194.344\cdot\frac{kip\cdot ft}{ft}$$

Load Case IV

Factored vertical force at the base of the footing

$$F_{VFtLC4StrI} := 1.25\cdot\left(DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing}\right) + 1.5DW_{Sup} \cdots$$
$$+ 1.35\cdot\left(EV_{earthFt} + EV_{earthBk}\right) + 1.75V_{LSFooting}$$

$$F_{VFtLC4StrI} = 53.013\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC4StrI} := 1.5\cdot P_{EHFooting} + 1.75P_{LSFooting} + 0.5TU = 19.855\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4StrI} := 1.25\cdot DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + 1.25DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ \left(1.25\cdot DC_{Sup} + 1.5\cdot DW_{Sup}\right)\cdot\left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ 1.5\cdot P_{EHFooting}\cdot\frac{\left(h_{backwall} + h_{wall} + t_{footing}\right)}{3} + 1.75V_{LSFooting}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ 1.75\cdot P_{LSFooting}\cdot\frac{\left(h_{backwall} + h_{wall} + t_{footing}\right)}{2} + 1.0\cdot EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ 1.35\cdot EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right) + 0.5\cdot TU\cdot\left(h_{wall} + t_{footing}\right)$$

$$M_{uFtLC4StrI} = 197.175\cdot\frac{kip\cdot ft}{ft}$$

Service I

Service I = 1.0DC + 1.0DW + 1.0LL + 1.0BR + 1.0WS + 1.0WL + 1.0EH + 1.0EV + 1.0LS + 1.0TU

Load Case I

Factored vertical force at the base of the footing

$$F_{VFtLC1SerI} := DC_{backwall} + DC_{wall} + DC_{footing} + EV_{earthBk} + EV_{earthFt} = 32.052\cdot\frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC1SerI} := P_{EHFooting} = 11.062\cdot\frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC1SerI} := DC_{backwall}\cdot\left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall}\cdot\left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ P_{EHFooting}\cdot\frac{\left(h_{backwall} + h_{wall} + t_{footing}\right)}{3} \cdots$$
$$+ EV_{earthBk}\cdot\left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt}\cdot\left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC1SerI} = 80.288\cdot\frac{kip\cdot ft}{ft}$$

Load Case III

Factored vertical force at the base of the footing

$$F_{VFtLC3SerI} := DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing} + DW_{Sup} + R_{LLFootingMax} \cdots + (EV_{earthFt} + EV_{earthBk})$$

$$F_{VFtLC3SerI} = 44.053 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC3SerI} := P_{EHFooting} = 11.062 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC3SerI} := DC_{backwall} \cdot \left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall} \cdot \left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ (DC_{Sup} + DW_{Sup} + R_{LLFootingMax}) \cdot \left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ P_{EHFooting} \cdot \frac{(h_{backwall} + h_{wall} + t_{footing})}{3} \cdots$$
$$+ EV_{earthBk} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt} \cdot \left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right)$$

$$M_{uFtLC3SerI} = 121.293 \cdot \frac{kip \cdot ft}{ft}$$

Load Case IV

Factored vertical force at the base of the footing

$$F_{VFtLC4SerI} := DC_{Sup} + DC_{backwall} + DC_{wall} + DC_{footing} + DW_{Sup} \cdots + EV_{earthFt} + EV_{earthBk} + V_{LSFooting}$$

$$F_{VFtLC4SerI} = 40.176 \cdot \frac{kip}{ft}$$

Factored shear force parallel to the transverse axis of the footing

$$V_{uFtLC4SerI} := P_{EHFooting} + P_{LSFooting} + TU = 13.124 \cdot \frac{kip}{ft}$$

Factored moment about the longitudinal axis of the footing

$$M_{uFtLC4SerI} := DC_{backwall} \cdot \left(l_{heel} + \frac{t_{backwall}}{2} - \frac{B_{footing}}{2}\right) + DC_{wall} \cdot \left(l_{heel} + \frac{t_{wall}}{2} - \frac{B_{footing}}{2}\right) \cdots$$
$$+ (DC_{Sup} + DW_{Sup}) \cdot \left(l_{heel} + l_{brtowall} - \frac{B_{footing}}{2}\right) + P_{EHFooting} \cdot \frac{(h_{backwall} + h_{wall} + t_{footing})}{3} \cdots$$
$$+ EV_{earthBk} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + EV_{earthFt} \cdot \left(\frac{B_{footing}}{2} - \frac{l_{toe}}{2}\right) \cdots$$
$$+ V_{LSFooting} \cdot \left(\frac{l_{heel}}{2} - \frac{B_{footing}}{2}\right) + P_{LSFooting} \cdot \frac{(h_{backwall} + h_{wall} + t_{footing})}{2} \cdots$$
$$+ TU \cdot (h_{wall} + t_{footing})$$

$$M_{uFtLC4SerI} = 126.982 \cdot \frac{kip \cdot ft}{ft}$$

Summary of Forces and Moments at the Base of the Footing

Factored vertical force, F_{VFt} (kip/ft)

	Strength I	Service I
LC I	41.85	32.05
LC III	59.80	44.05
LC IV	53.01	40.18

Factored shear force parallel to the transverse axis of the footing, V_{uFt} (kip/ft)

	Strength I	Service I
LC I	16.59	11.06
LC III	16.59	11.06
LC IV	19.85	13.12

Factored moment about the longitudinal axis of the footing, M_{uFt} (kip·ft/ft)

	Strength I	Service I
LC I	133.01	80.29
LC III	194.34	121.29
LC IV	197.18	126.98

Step 4.6 Pile Size and Layout Design

Description

This step presents the pile type, size, and preliminary layout.

This example uses steel H piles since it is the most commonly used pile type in Michigan.

Typically, pile type is selected after evaluating other possibilities, such as ground improvement techniques, other foundation types, and constructability.

Pile embedment into the footing $Pile\_embd := 6in$ **BDM 7.03.09.A5**

Note: A tremie seal is not used for this footing. If a tremie seal is used, the pile embedment into the footing is 1 ft. A tremie seal design is given in Appendix 4.A.

The following parameters are considered to determine the pile layout:

1. Pile spacing: The depth of commonly used H-piles ranges from 10 to 14 inches. The minimum pile spacing is controlled by the greater of 30 inches or 2.5 times the pile diameter. As a practice, MDOT uses 3 times the pile diameter as the spacing. **LRFD 10.7.1.2**

   Selected pile section HP 12X53

   Minimum spacing $Spacing_{min} := 36in$

2. Edge distance: The usual minimum edge distance for piles is 18 inches. **BDM 7.03.09.A7**

   Pile edge distance $PileEdgeDist := 18in$

Use two rows of piles.

Maximum number of piles in each row the footing can accommodate

$$N_{MaxPiles} := \frac{L_{footing} - 2 \cdot PileEdgeDist}{Spacing_{min}} = 20.917$$

Spacing between two rows

$$S_B := B_{footing} - 2 \cdot PileEdgeDist = 8\,ft$$

The loads acting on the two rows of piles are determined as follows:

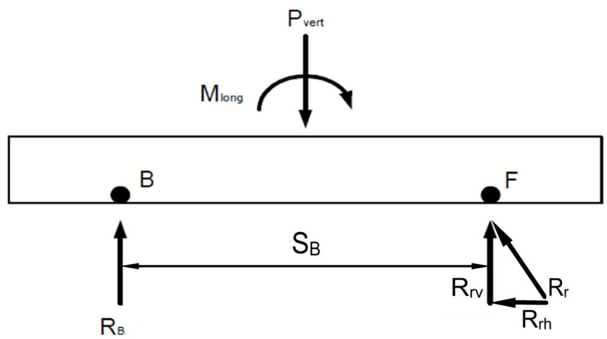
Load Case III under Strength I limit state is identified as the governing load case by examining the summary tables presented at the end of Step 4.5. Therefore:

Total vertical force

$$P_{vert} := F_{VFtLC3StrI} \cdot L_{footing} = 3.932 \times 10^{3} \cdot kip$$

Total moment about the longitudinal axis of the footing

$$M_{long} := M_{uFtLC3StrI} \cdot L_{footing} = 1.278 \times 10^{4} \cdot kip \cdot ft$$



Total vertical loads on each row of piles are calculated based on static equilibrium.

$$R_{rv} := \frac{\left(M_{long} + P_{vert} \cdot \frac{S_B}{2}\right)}{S_B} = 3.563 \times 10^{3} \cdot kip \qquad R_B := P_{vert} - R_{rv} = 368.598 \cdot kip$$

Selected batter for the front row piles 3V:1H **BDM 7.03.09.A9**

$$Pile_{batter} := \frac{3}{1}$$

Total axial load on the front row piles $R_r := R_{rv} \cdot \frac{\sqrt{Pile_{batter}^{\ 2} + 1^2}}{Pile_{batter}} = 3.756 \times 10^3 \cdot kip$

Consult the Geotechnical Services Section to select a nominal pile resistance for the selected section. **BDM 7.03.09.B**

Nominal pile resistance $R_n := 350kip$

Assume that the Nominal Pile Driving Resistance (R_{ndr}) is verified using the FHWA-modified Gates Dynamic Formula, and no PDA test or static load tests are performed

The resistance factor for driven piles $\varphi_{dyn} := 0.5$ **BDM 7.03.09.B2**

Factored nominal pile resistance $R_R := \varphi_{dyn} \cdot R_n = 175 \cdot kip$

Required number of piles in the front row $n_{front\_required} := \frac{R_r}{R_R} = 21.462$ $N_{MaxPiles} = 20.917$

Check if the required number of piles in the front row exceeds the maximum number of piles the footing can accommodate $Check := if\left(n_{front\_required} < N_{MaxPiles}, \text{"OK"}, \text{"Not OK"}\right) = \text{"Not OK"}$

The required number of piles in the front row is greater than the maximum number of piles that can be accommodated within the selected footing dimensions. Consider a larger pile section.

Selected pile section HP 14X73

Consult the Geotechnical Services Section to select a nominal pile resistance for the selected section. **BDM 7.03.09.B**

Nominal pile resistance $R_n := 500kip$ **BDM 7.03.09.B**

Factored nominal pile resistance $R_R := \varphi_{dyn} \cdot R_n = 250 \cdot kip$

Required number of piles in the front row $n_{front} := \frac{R_r}{R_R} = 15.023$

$$n_{front} := trunc\left(n_{front}\right) + 1 = 16$$

Pile spacing $spacing := \frac{\left(L_{footing} - 2 \cdot PileEdgeDist\right)}{n_{front} - 1} = 50.2 \cdot in$

Number of pile selected for the front row $N_{front} := 16$

Selected pile spacing $FrontPileSpacing := 50in$

Select 16 front row piles spaced at 4 ft-2 in., max = 62 ft 9 in.

Flange width and depth of the pile $b_f := 14.585in$ $d_{pile} := 13.61in$

Check if the spacing of the piles is greater than 3D $Check := if\left(FrontPileSpacing > 3d_{pile}, "OK", "Not OK"\right) = "OK"$

Piles in the Back Row

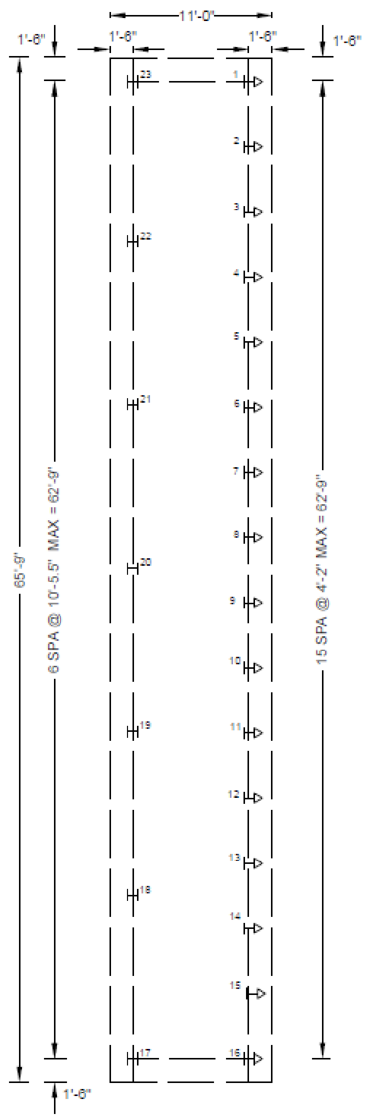
Number of pile selected for the back row $N_{back} := 7$

Selected pile spacing $BackPileSpacing := 10ft + 5.5in = 125.5 \cdot in$

Select 7 back row piles spaced at 10 ft 5.5 in.

Note: As per the AASHTO LRFD Article 10.7.5, the effects of corrosion and deterioration from environmental conditions shall be considered in the selection of the pile type and cross-section. The soil conditions should be examined to ensure that there is no indication of contamination that would cause piles to corrode. Consult the Geotechnical Services Section to determine a suitable pile type and cross-section for the selected site.

The preliminary pile layout is shown in the following figure.



Step 4.7 Pile Capacity Check

Description

This step presents the pile axial and lateral load calculations and pile capacity checks.

Strength I Limit State

Load Case I

Total vertical load $P_{vert} := F_{VFtLC1StrI} \cdot L_{footing} = 2.751 \times 10^3 \cdot kip$

Total shear force parallel to the transverse axis of the footing $P_{lat} := V_{uFtLC1StrI} \cdot L_{footing} = 1.091 \times 10^3 \cdot kip$

Total moment about the longitudinal axis of the footing $M_{long} := M_{uFtLC1StrI} L_{footing} = 8.745 \times 10^3 \cdot kip \cdot ft$

The total vertical loads on the front and back row of piles are calculated based on the static equilibrium of the footing.

$$R_{rv} := \frac{\left(M_{long} + P_{vert} \cdot \frac{S_B}{2}\right)}{S_B} = 2.469 \times 10^3 \cdot kip \qquad R_B := P_{vert} - R_{rv} = 282.533 \cdot kip$$

Vertical component of the axial force on a front row battered pile $R_{rv\_SingleLC1StrI} := \frac{R_{rv}}{n_{front}} = 154.304 \cdot kip$

Axial force on a front row battered pile $R_{r\_SingleLC1StrI} := R_{rv\_SingleLC1StrI} \cdot \frac{\sqrt{Pile_{batter}^2 + 1^2}}{Pile_{batter}} = 162.65 \cdot kip$

Pile section selected in Step 4.6 HP 14X73

Factored nominal pile resistance $R_R = 250 \cdot kip$

Check if R_R > Axial force $Check := if\left(R_R > R_{r\_SingleLC1StrI}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Horizontal component of the axial force on a front row battered pile $R_{rh\_SingleLC1StrI} := \frac{R_{r\_SingleLC1StrI}}{\sqrt{Pile_{batter}^2 + 1^2}} = 51.435 \cdot kip$

Total lateral force resisted by the battered piles (i.e. the horizontal component of the axial force) $P_{HBatteredPiles} := R_{rh\_SingleLC1StrI} \cdot n_{front} = 822.952 \cdot kip$

Check if the lateral load capacity of the battered piles is greater than the lateral load demand **BDM 7.03.09.A.11**

$$Check := if\left(P_{HBatteredPiles} > P_{lat}, \text{"Yes"}, \text{"No, check if the vertical piles can resist the remaining later load"}\right)$$

$$Check = \text{"No, check if the vertical piles can resist the remaining later load"}$$

Note: Per MDOT practice, the typical lateral resistance of a vertical pile is 12 kips. A pile bending (p-y) analysis may be performed by incorporating soil-pile interaction to determine the more accurate lateral load resistance of the piles. Consult the Geotechnical Services Section for more information.

The required lateral load resistance of a pile $P_{latReqd\_LC1StrI} := \frac{\left(P_{lat} - P_{HBatteredPiles}\right)}{\left(N_{front} + N_{back}\right)} = 11.653 \cdot kip$

Lateral load resistance of a pile (from the Geotechnical Services Section)

$P_{latProvided} := 12kip$

Check if the lateral load resistance > the required lateral load resistance

$Check := if(P_{latProvided} > P_{latReqd\_LC1StrI}, "OK", "Not OK") = "OK"$

Vertical force on a back row pile

$$R_{B\_SingleLC1StrI} := \frac{R_B}{N_{back}} = 40.362 \cdot kip$$

Check if R_R > Axial force

$Check := if(R_R > R_{B\_SingleLC1StrI}, "OK", "Not OK") = "OK"$

Load Case III

Total vertical load

$P_{vert} := F_{VFtLC3StrI} \cdot L_{footing} = 3.932 \times 10^3 \cdot kip$

Total shear force parallel to the transverse axis of the footing

$P_{lat} := V_{uFtLC3StrI} \cdot L_{footing} = 1.091 \times 10^3 \cdot kip$

Total moment about the longitudinal axis of the footing

$M_{long} := M_{uFtLC3StrI} L_{footing} = 1.278 \times 10^4 \cdot kip \cdot ft$

The total vertical loads on the front and back row of piles are calculated based on static equilibrium of the footing.

$$R_{rv} := \frac{\left(M_{long} + P_{vert} \cdot \frac{S_B}{2}\right)}{S_B} = 3.563 \times 10^3 \cdot kip \qquad R_B := P_{vert} - R_{rv} = 368.598 \cdot kip$$

Vertical component of the axial force on a front row battered pile

$$R_{rv\_SingleLC3StrI} := \frac{R_{rv}}{n_{front}} = 222.695 \cdot kip$$

Axial force on a front row battered pile

$$R_{r\_SingleLC3StrI} := R_{rv\_SingleLC3StrI} \cdot \frac{\sqrt{Pile_{batter}^2 + 1^2}}{Pile_{batter}} = 234.741 \cdot kip$$

Check if R_R > Axial force

$Check := if(R_R > R_{r\_SingleLC3StrI}, "OK", "Not OK") = "OK"$

Horizontal component of the axial force on a front row battered pile

$$R_{rh\_SingleLC3StrI} := \frac{R_{r\_SingleLC3StrI}}{\sqrt{Pile_{batter}^2 + 1^2}} = 74.232 \cdot kip$$

Total lateral force resisted by the battered piles

$P_{HBatteredPiles} := R_{rh\_SingleLC3StrI} \cdot n_{front} = 1.188 \times 10^3 \cdot kip$

Check if the lateral load capacity of the battered piles is greater than the lateral load demand **BDM 7.03.09.A.11**

$Check := if(P_{HBatteredPiles} > P_{lat}, "Yes", "No, check if the vertical piles can resist the remaining later load") = "Yes"$

The required lateral load resistance of a pile

$P_{latReqd\_LC3StrI} := 0$

Vertical force on a back row pile

$$R_{B\_SingleLC3StrI} := \frac{R_B}{N_{back}} = 52.657 \cdot kip$$

Check if R_R > Axial force

$Check := if(R_R > R_{B\_SingleLC3StrI}, "OK", "Not OK") = "OK"$

Load Case IV

Total vertical load — $P_{vert} := F_{VFtLC4StrI} \cdot L_{footing} = 3.486 \times 10^{3} \cdot kip$

Total shear force parallel to the transverse axis of the footing — $P_{lat} := V_{uFtLC4StrI} \cdot L_{footing} = 1.305 \times 10^{3} \cdot kip$

Total moment about the longitudinal axis of the footing — $M_{long} := M_{uFtLC4StrI} L_{footing} = 1.296 \times 10^{4} \cdot kip \cdot ft$

The total vertical loads on the front and back row of piles are calculated based the static equilibrium of the footing.

$$R_{rv} := \frac{\left(M_{long} + P_{vert} \cdot \frac{S_B}{2}\right)}{S_B} = 3.363 \times 10^{3} \cdot kip \qquad R_B := P_{vert} - R_{rv} = 122.275 \cdot kip$$

Vertical component of the axial force on a front row battered pile — $R_{rv\_SingleLC4StrI} := \frac{R_{rv}}{n_{front}} = 210.209 \cdot kip$

Axial force on a front row battered pile — $R_{r\_SingleLC4StrI} := R_{rv\_SingleLC4StrI} \cdot \frac{\sqrt{Pile_{batter}^{2} + 1^{2}}}{Pile_{batter}} = 221.58 \cdot kip$

Check if $R_R >$ Axial force — $Check := if\left(R_R > R_{r\_SingleLC4StrI}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Horizontal component of the axial force on a front row battered pile — $R_{rh\_SingleLC4StrI} := \frac{R_{r\_SingleLC4StrI}}{\sqrt{Pile_{batter}^{2} + 1^{2}}} = 70.07 \cdot kip$

Total lateral force resisted by the battered piles — $P_{HBatteredPiles} := R_{rh\_SingleLC4StrI} \cdot n_{front} = 1.121 \times 10^{3} \cdot kip$

Check if the lateral load capacity of the battered piles is greater than the lateral load demand **BDM 7.03.09.A.11**

$Check := if\left(P_{HBatteredPiles} > P_{lat}, \text{"Yes"}, \text{"No, check if the vertical piles can resist the remaining later load"}\right)$

$Check = \text{"No, check if the vertical piles can resist the remaining later load"}$

The required lateral load resistance of a pile — $P_{latReqd\_LC4StrI} := \frac{\left(P_{lat} - P_{HBatteredPiles}\right)}{\left(N_{front} + N_{back}\right)} = 8.015 \cdot kip$

Lateral load resistance of a pile (from the Geotechnical Services Section) — $P_{latProvided} = 12 \cdot kip$

Check if the lateral load resistance > the required lateral load resistance — $Check := if\left(P_{latProvided} > P_{latReqd\_LC1StrI}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Vertical force on a back row pile — $R_{B\_SingleLC4StrI} := \frac{R_B}{N_{back}} = 17.468 \cdot kip$

Check if $R_R >$ Axial force — $Check := if\left(R_R > R_{B\_SingleLC4StrI}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Summary of Forces Acting on a Single Pile

A summary of forces acting on a single pile under different Strength I limit state load cases is listed in the following tables. The pile penetration depth design, driveability analysis, and settlement analysis are performed by the geotechnical engineers to evaluate the adequacy of the selected design.

Axial force on a battered pile in the front row (kip)

	Strength I
LC I	162.65
LC III	234.74
LC IV	221.58

Axial force on a pile in the back row (kip)

	Strength I
LC I	40.36
LC III	52.66
LC IV	17.47

Required lateral force resistance of a pile (kip)

	Strength I
LC I	11.65
LC III	0.00
LC IV	8.02

Step 4.8 Backwall Design

Description

Since the backwall forces and moments used in Step 2.7 are not impacted by the use of piles, please refer to the design calculations and details presented in Step 2.7.

Step 4.9 Abutment Wall Design

Description

Since the abutment wall forces and moments used in Step 2.8 are not impacted by the use of piles, please refer to the design calculations and details presented in Step 2.8.

Step 4.10 Structural Design of the Footing

Description

This step presents the structural design of the abutment footing.

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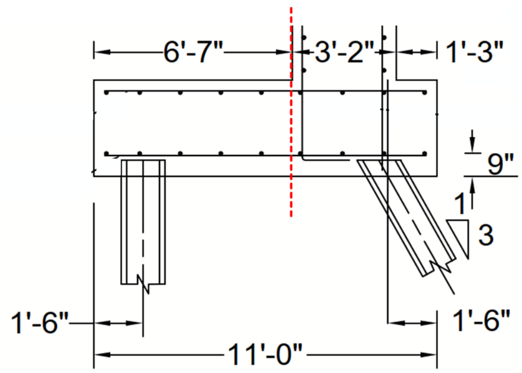
Note: In this example, the length of the footing and the abutment wall are 65.75 ft and 63.75 ft, respectively. Since the cantilevered length of the footing in the longitudinal direction is limited to 1 ft on each side, the shear and moment acting on the footing in the longitudinal direction are small and do not require flexural and shear designs.

Design for Flexure

In a typical spread footing, the critical section for flexure, due to the loads acting on the toe, is at the front face of the wall. As shown below, since the front row piles are located behind the critical section, the flexural capacity of the pile cap is not evaluated.

For the heel of the footing, the critical section is located at the back face of the wall. The footing is designed considering the flexural demand at this section since the back row of piles is located closer to the edge of the heel.



As per the summary tables presented at the end of Step 4.7, the maximum and minimum Strength I limit state axial forces at the back row piles are from LC III and LC IV, respectively.

The maximum total axial load on the back row piles $R_{backMax} := R_{B\_SingleLC3StrI} \cdot N_{back} = 368.598 \cdot kip$

The minimum total axial load on the back row piles $R_{backMin} := R_{B\_SingleLC4StrI} \cdot N_{back} = 122.275 \cdot kip$

Distance from center of the pile to the critical section $d_{arm} := l_{heel} - PileEdgeDist = 5.083\ ft$

Bottom of the footing at the critical section

The tension at the bottom of the footing is developed due to the axial forces at the back row piles. The selfweight of the footing and the vertical earth load on the heel develop the resisting moment. For a safe design, the maximum axial load at the back row piles is used with the minimum load factors for the resisting forces.

The minimum load factors for dead load and earth load are 0.9 and 1.0, respectively. **LRFD Table 3.4.1-2**

The maximum moment at the critical section to develop tension at the bottom of the footing

$$M_{rb} := \frac{R_{backMax} \cdot d_{arm}}{L_{footing}} - 0.9W_c \cdot t_{footing} \cdot \frac{l_{heel}^2}{2} - EV_{earthBk} \cdot \frac{l_{heel}}{2} = -36.942 \cdot \frac{kip \cdot ft}{ft}$$

The negative moment does not require a design for the flexural reinforcement at the bottom of the footing.

Top of the footing at the critical section

Load case IV under the Strength I limit state develops the minimum axial force at the back row piles. The same load case develops the maximum vertical downward loads on the heel due to self weight of the footing, vertical earth load, and live load surcharge.

The maximum moment at the critical section to develop tension at the top of the footing

$$M_{rt} := 1.25W_c \cdot t_{footing} \cdot \frac{l_{heel}^2}{2} + 1.35EV_{earthBk} \cdot \frac{l_{heel}}{2} + 1.75V_{LSFooting} \cdot \frac{l_{heel}}{2} - \frac{R_{backMin} \cdot d_{arm}}{L_{footing}} = 88.333 \cdot \frac{kip \cdot ft}{ft}$$

Flexure Resistance

LRFD 5.6.3.2

The design procedure consists of calculating the reinforcing steel area required to satisfy the moment demand, and checking the selected steel area against the requirements and limitations for developing an adequate moment capacity, controlling crack width, and managing shrinkage and temperature stresses.

As a trial, select No.8 bars. $bar := 8$

Nominal diameter of a reinforcing steel bar $d_{bar} := Dia(bar) = 1 \cdot in$

Cross-section area of a reinforcing steel bar on the flexural tension side $A_{bar} := Area(bar) = 0.79 \cdot in^2$

The spacing shall not exceed 12 in. when the thickness of the footing is greater than 18 in. **LRFD 5.10.6**

Footing thickness $t_{footing} = 3\,ft$

Selected spacing of reinforcing bars $s_{bar} := 12 \cdot in$

Area of tension steel in a 1 ft wide strip $A_{sProvided} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.79 \cdot in^2$

Effective depth $d_e := t_{footing} - Cover_{ft} = 32 \cdot in$

Resistance factor for flexure $\phi_f := 0.9$ **LRFD 5.5.4.2**

A 1-ft wide strip is selected for the design.

Width of the compression face of the section $b := 12in$

Stress block factor $\beta_1 := \min\left[\max\left[0.85 - 0.05 \cdot \left(\frac{f_c - 4ksi}{ksi}\right), 0.65\right], 0.85\right] = 0.85$ **LRFD 5.6.2.2**

Solve the following quadratic equation to calculate the required A_s needed to satisfy the moment demand. Use an assumed initial A_s value to solve the quadratic equation.

The initial assumption for A_s $A_s := 1in^2$

Solve the quadratic equation for the area of steel required

$$\text{Given} \quad M_{rt} \cdot ft = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$$

Required steel area $A_{sRequired} := Find(A_s) = 0.625 \cdot in^2$

Check if $A_{sProvided} > A_{sRequired}$ $Check := if(A_{sProvided} > A_{sRequired}, "OK", "Not OK") = "OK"$

Moment capacity of the section with the provided steel

$$M_{Provided} := \phi_f \cdot A_{sProvided} \cdot f_y \cdot \frac{\left[d_e - \frac{1}{2} \cdot \left(\frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]}{ft}$$

$$M_{Provided} = 111.007 \cdot \frac{kip \cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis

$$c := \frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.82 \cdot in$$

Check the validity of the assumption, $f_s = f_y$

$$Check\_f_s := if\left(\frac{c}{d_e} < 0.6, "OK", "Not OK"\right) = "OK"$$

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must develop a factored flexural resistance equal to the lesser of the cracking moment or 1.33 times the factored moment from the applicable strength limit state load combinations.

Flexural cracking variability factor $\gamma_1 := 1.6$ For concrete structures that are not precast segmental

Ratio of specified minimum yield strength to ultimate tensile strength of the nonprestressed reinforcement $\gamma_3 := 0.67$ For ASTM615 grade 60 reinforcement

Section modulus

$$S_c := \frac{1}{6} \cdot b \cdot t_{footing}^2 = 2.592 \times 10^3 \cdot in^3$$

Cracking moment

$$M_{cr} := \frac{\gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c}{ft} = 96.254 \cdot \frac{kip \cdot ft}{ft}$$

1.33 times the factored moment demand

$$1.33 \cdot M_{rt} = 117.482 \cdot \frac{kip \cdot ft}{ft}$$

Required moment to satisfy the minimum reinforcement requirement

$$M_{req} := min(1.33M_{rt}, M_{cr}) = 96.254 \cdot \frac{kip \cdot ft}{ft}$$

Check the adequacy of section capacity

$$Check := if(M_{Provided} > M_{req}, "OK", "Not OK") = "OK"$$

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

Concrete is subjected to cracking. Limiting the width of expected cracks under service conditions extends the service life. The width of potential cracks can be minimized through proper placement of the reinforcement. The check for crack control requires that the actual stress in the reinforcement should not exceed the service limit state stress.

The spacing requirement for the mild steel reinforcement in the layer closer to the tension face

$$s \le \frac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$$

LRFD Eq. 5.6.7-1

Exposure factor for Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest bar $d_c := Cover_{ft} = 4 \cdot in$

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer nearest the tension face $\beta_s := 1 + \dfrac{d_c}{0.7(t_{footing} - d_c)} = 1.179$

The calculation of tensile stress in nonprestressed reinforcement at the service limit state, f_{ss}, requires establishing the neutral axis location and the moment demand at the critical section.

The position of the section's neutral axis is determined through an iterative process to calculate the actual stress in the reinforcement. This process starts with an assumed position of the neutral axis.

Assumed position of the neutral axis $x := 5 \cdot in$

Given $\dfrac{1}{2} \cdot b \cdot x^2 = \dfrac{E_s}{E_c} \cdot A_{sProvided} \cdot (d_e - x)$

Position of the neutral axis $x_{na} := Find(x) = 5.303 \cdot in$

The axial force in the back row piles (R_B) from Load Case IV under Service I limit state is calculated as shown below.

Total vertical load $P_{vert} := F_{VFtLC4SerI} \cdot L_{footing} = 2.642 \times 10^3 \cdot kip$

Total shear force parallel to the transverse axis of the footing $P_{lat} := V_{uFtLC4SerI} \cdot L_{footing} = 862.914 \cdot kip$

Total moment about the longitudinal axis of the footing $M_{long} := M_{uFtLC4SerI} L_{footing} = 8.349 \times 10^3 \cdot kip \cdot ft$

The total vertical loads on the front and back row of piles are calculated based on static equilibrium of the footing.

$$R_{rv} := \frac{\left(M_{long} + P_{vert} \cdot \dfrac{S_B}{2}\right)}{S_B} = 2.364 \times 10^3 \cdot kip \qquad R_B := P_{vert} - R_{rv} = 277.156 \cdot kip$$

The moment at the critical section under Service I limit state that generates tension at the top of the footing is:

$$M_{heelTopSerI} := W_c \cdot t_{footing} \cdot \frac{l_{heel}^2}{2} + EV_{earthBk} \cdot \frac{l_{heel}}{2} + V_{LSFooting} \cdot \frac{l_{heel}}{2} - \frac{R_B \cdot d_{arm}}{L_{footing}} = 50.188 \cdot \frac{kip \cdot ft}{ft}$$

Tensile force in the reinforcing steel due to the Service I limit state moment $T_s := \dfrac{M_{heelTopSerI}}{d_e - \dfrac{x_{na}}{3}} \cdot ft = 19.9 \cdot kip$

Stress in the reinforcing steel due to the Service I limit state moment $f_{ss1} := \dfrac{T_s}{A_{sProvided}} = 25.216 \cdot ksi$

f_{ss} not exceed $0.6f_y$ $f_{ss} := \min(f_{ss1}, 0.6f_y) = 25.216 \cdot ksi$

Required reinforcement spacing

$$bar_{spaReq} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 15.554 \cdot in$$

Check if the spacing provided < the required spacing

$$Check := if\left(s_{bar} < bar_{spaReq}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shrinkage and Temperature Reinforcement

LRFD 5.10.6

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the backwall.

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := \min\left[\begin{bmatrix} \left(0.60 \frac{in^2}{ft}\right) \\ \max\left[\begin{bmatrix} \left(0.11 \frac{in^2}{ft}\right) \\ \left[\frac{1.3 \cdot B_{footing} \cdot t_{footing} \cdot \frac{kip}{in \cdot ft}}{2\left(B_{footing} + t_{footing}\right) \cdot f_y}\right] \end{bmatrix}\right] \end{bmatrix}\right] \cdot ft = 0.306 \cdot in^2$$

Check if the provided area of steel > the required area of shrinkage and temperature steel

$$Check := if\left(A_{sProvided} > A_{shrink.temp}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Design for Shear

Shear design in abutment footings supported by piles consists of providing adequate resistance against one-way action (beam action shear) and two-way action (punching shear). For both one-way and two-way actions, the design shear is taken at the critical section. For abutment footings, one-way action is checked at the toe and heel. The factored shear force at the critical section is computed by cutting the footing at the critical section and taking the summation of the pile loads or portions of pile loads that are outside the critical section. Two-way action in abutment footings supported by piles is generally checked by taking a critical perimeter around individual piles or around a group of piles when the critical perimeter of individual piles overlap.

One-way shear

For one-way shear on the toe side, since the front row of piles are inside the critical section, a shear check at the critical section or towards the toe is not required.
On the heel side, the downward load is larger than the upward back row pile axial force. So, the top of the heel is in tension. Therefore, the critical section for shear is taken at the back face of the abutment wall.

LRFD C5.12.8.6.1

Effective width of the section

$$b = 12 \cdot in$$

Provided flexural reinforcement area

$$A_{sProvided} = 0.79 \cdot in^2$$

Depth of equivalent rectangular stress block

$$a := \frac{A_{sProvided} \cdot f_y}{0.85 \cdot f_c \cdot b} = 1.549 \cdot in$$

Effective shear depth

$$d_v := \max\left(d_e - \frac{a}{2}, 0.9 \cdot d_e, 0.72 \cdot t_{footing}\right) = 31.225 \cdot in$$

LRFD 5.7.2.8

The factored shear demand at the critical section for shear:

$$V_{uFtHeel} := 1.25 \cdot W_c \cdot t_{footing} \cdot l_{heel} + 1.35EV_{earthBk} + 1.75V_{LSFooting} - \frac{R_{backMin}}{L_{footing}} = 27.847 \cdot \frac{kip}{ft}$$

For a concrete footing in which the distance from point of zero shear to the face of the wall is less than $3d_v$, the simplified procedure for nonprestressed sections can be used. **LRFD 5.7.3.4.1**

Check if the distance l_{heel} is less than $3d_v$ $\quad$ $Check := if(l_{heel} < 3 \cdot d_v, "Yes", "No") = "Yes"$

Therefore, the simplified procedure is used.

Factor indicating the ability of diagonally cracked concrete to transmit tension and shear $\quad$ $\beta := 2$

Nominal shear resistance of concrete, V_n, is calculated as follows:

$$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_e = 42 \cdot kip$$ **LRFD Eq. 5.7.3.3-3**

$$V_{c2} := 0.25f_c \cdot b \cdot d_e = 288 \cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := \min(V_{c1}, V_{c2}) = 42.035 \cdot kip$$

Resistance factor for shear $\quad$ $\phi_v := 0.9$ **LRFD 5.5.4.2**

Factored shear resistance $\quad$ $V_r := \phi_v \cdot V_n = 37.831 \cdot kip$

Check if the factored shear resistance is greater than the shear demand $\quad$ $Check := if\left(\frac{V_r}{ft} > V_{uFtHeel}, "OK", "Not OK"\right) = "OK"$

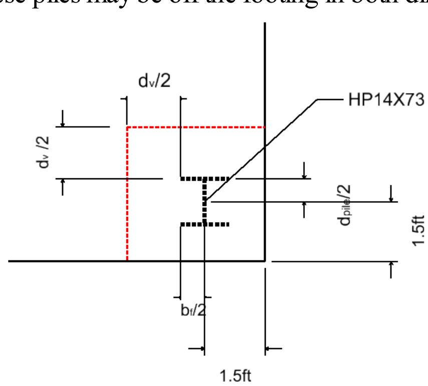
Two-way shear

For two-way shear, the critical perimeter of a pile, b_o, is located at a minimum of $0.5d_v$ from the perimeter of the pile. If portions of the critical perimeter are located off the footing, that portion of the critical perimeter is limited by the footing edge. **LRFD 5.12.8.6.3**

Pile section selected in Step 4.6 $\quad$ HP 14X73

Flange width and depth of the pile $\quad$ $b_f := 14.585in$ $\quad$ $d_{pile} := 13.61in$

The loads on the front piles are assumed to be identical. The critical case for two-way shear is the piles at the end of the front row since the critical perimeter of these piles may be off the footing in both directions.



Check if the critical perimeter is off the footing in the footing width direction

$$OffFooting1 := if\left(\frac{d_{pile}}{2} + \frac{d_v}{2} > PileEdgeDist, "Yes", "No"\right)$$

$$OffFooting1 = "Yes"$$

Check if the critical perimeter is off the footing in the footing length direction

$$OffFooting2 := if\left(\frac{b_f}{2} + \frac{d_v}{2} > PileEdgeDist, "Yes", "No"\right)$$

$$OffFooting2 = "Yes"$$

Length of the critical perimeter side parallel to the footing width direction

$$b_{01} := if\left(OffFooting1 = "Yes", \frac{d_{pile}}{2} + \frac{d_v}{2} + PileEdgeDist, d_{pile} + d_v\right)$$

$$b_{01} = 3.368\ ft$$

Length of the critical perimeter side parallel to the footing length direction

$$b_{02} := if\left(OffFooting2 = "Yes", \frac{b_f}{2} + \frac{d_v}{2} + PileEdgeDist, b_f + d_v\right)$$

$$b_{02} = 3.409\ ft$$

Critical perimeter

$$b_0 := 2 \cdot \left(b_{01} + b_{02}\right) = 13.554\ ft$$

Ratio of long side to short side of the rectangle through which the concentrated load is transmitted

$$\beta_c := \frac{b_f}{d_{pile}} = 1.072$$

Nominal shear resistance

$$V_{n1\_2way} := \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f_c \cdot ksi} \cdot b_0 \cdot d_v = 1.588 \times 10^3 \cdot kip$$

$$V_{n2\_2way} := 0.126 \cdot \left(\sqrt{f_c \cdot ksi} \cdot b_0 \cdot d_v\right) = 1.108 \times 10^3 \cdot kip$$

LRFD Eq. 5.12.8.6.3-1

$$V_{n\_2way} := \min\left(V_{n1\_2way}, V_{n2\_2way}\right) = 1.108 \times 10^3 \cdot kip$$

Factored shear resistance

$$V_{r\_2way} := \phi_v \cdot V_{n\_2way} = 997.531 \cdot kip$$

According to the summary tables of the pile axial forces, LC III under Strength I limit state developed the maximum pile axial force at the front row piles.

Maximum two-way shear demand

$$R_{r\_SingleLC3StrI} = 234.741 \cdot kip$$

Check if the factored shear resistance is greater than the maximum pile axial force

$$Check := if\left(V_{r\_2way} > R_{r\_SingleLC3StrI}, "OK", "Not\ OK"\right) = "OK"$$

Development Length of Reinforcement

The flexural reinforcing steel must be developed on each side of the critical section for its full development length.

LRFD 5.10.8.1.2

Available development length

$$l_{d.available} := l_{heel} - Cover_{ft} = 6.25\ ft$$

Basic development length

$$l_{db} := 2.4 \cdot d_{bar} \cdot \frac{f_y}{\sqrt{f_c \cdot ksi}} = 6.928\ ft$$

LRFD Eq. 5.10.8.2.1a-2

Reinforcement location factor	$\lambda_{rl} := 1.3$	More than 12 in. concrete below	
Coating factor	$\lambda_{cf} := 1.5$	Epoxy coated bars with less than $3d_b$ cover	
Reinforcement confinement factor	$\lambda_{rc} := 0.4$	For $C_b > 2.5$ in. and No. 8 bars or smaller	
Excess reinforcement factor	$\lambda_{er} := \frac{A_{sRequired}}{A_{sProvided}} = 0.792$		**LRFD Eq. 5.10.8.2.1c-4**
For normal weight concrete	$\lambda := 1$		
Required development length	$l_{d.required} := l_{db} \cdot \frac{(\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er})}{\lambda} = 4.278\ ft$		**LRFD Eq. 5.10.8.2.1a-1**
Check if $l_{d.available} > l_{d.required}$	Check := if($l_{d.available} > l_{d.required}$, "OK" , "Not OK") = "OK"		

Shrinkage and Temperature Reinforcement

The following calculations check the required amount of reinforcing steel in the secondary direction to control shrinkage and temperature stresses in the footing.

The reinforcement along the longitudinal direction of the footing at the top and bottom should satisfy the shrinkage and temperature reinforcement requirement. **LRFD 5.10.6**

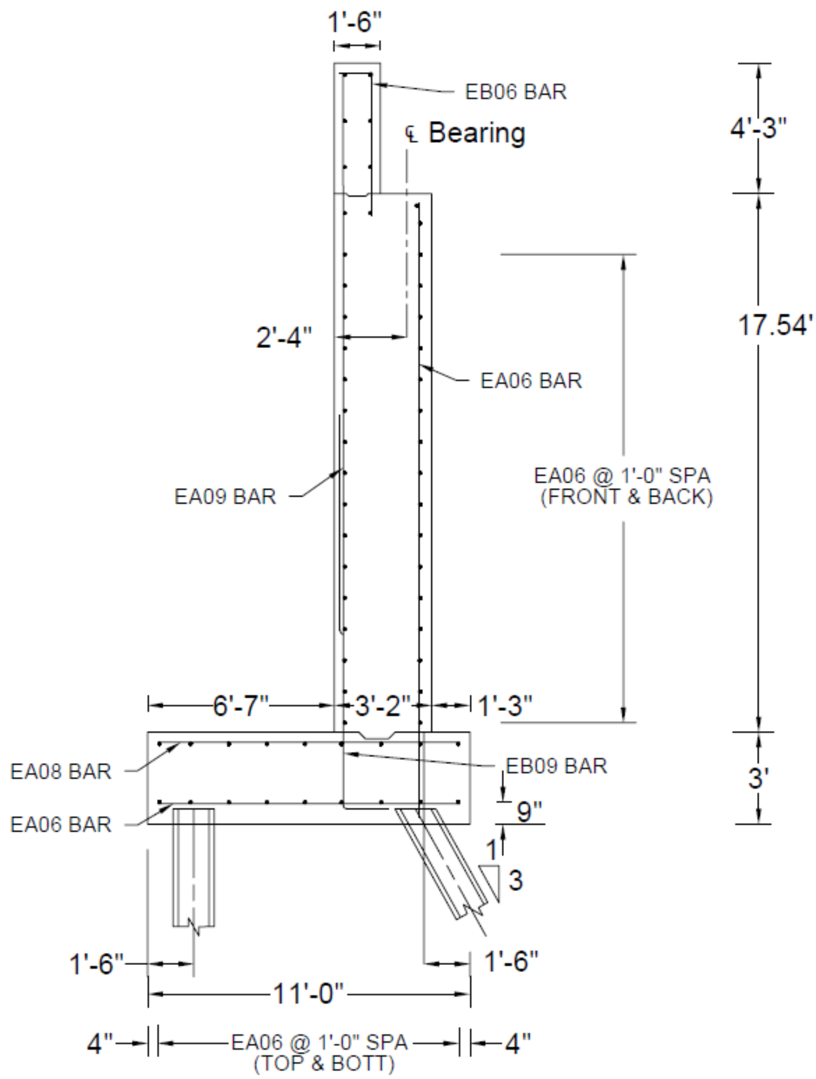
The spacing of reinforcement shall not exceed 12 in. since the footing thickness is greater than 18 in. **LRFD 5.10.6**

MDOT practice is to use 18 in. as the maximum spacing. **BDG 5.16.01**

As a trial, select No. 6 bars.	bar := 6
Nominal diameter of a reinforcing steel bar	$d_{bST} := \text{Dia}(bar) = 0.75 \cdot in$
Area of a reinforcing steel bar	$A_{barST} := \text{Area}(bar) = 0.44 \cdot in^2$
Selected bar spacing	$s_{barST} := 12 \cdot in$
Provided horizontal reinforcement area	$A_{sProvidedST} := \frac{A_{barST} \cdot 12in}{s_{barST}} = 0.44 \cdot in^2$
Required minimum area of shrinkage and temperature reinforcement in the the footing	$A_{shrink.temp} = 0.306 \cdot in^2$
Check if the area of steel provided > the required area of shrinkage and temperature steel	Check := if($A_{sProvidedST} > A_{shrink.temp}$, "OK" , "Not OK") = "OK"

The footing design presented in this step provides the following details:

- No. 8 bars at 12.0 in. spacing ($A_s = 0.79$ in.^{2}/ft) as the transverse flexural reinforcement at the top of the footing.
- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the transverse shrinkage and temperature reinforcement at the bottom of the footing.
- No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the longitudinal shrinkage and temperature reinforcement at the top and bottom of the footing.



Note: Refer to MDOT Bridge Design Guides for additional bars, laps, embedment, keyway dimensions, and drainage details. They are not shown in this drawing for clarity of main reinforcement.

Appendix 4.A Tremie Seal Design

Description

This appendix presents the design of a tremie seal.

Generally, tremie seals should be used on all structures when the pumping of water down below the bottom of footing is expected to be challenging. **BDM 7.03.06**

A tremie seal shall be designed to resist the hydrostatic pressure at the bottom of the tremie by a combination of its weight and the bond between the cofferdam and piles. The allowable bond strength between the tremie and piles is 10 psi. The allowable bond strength between the tremie and the cofferdam is 5 psi. The allowable tension from the bending of the tremie seal is 30 psi. **BDM 7.03.06A**

The design of a tremie seal consists of determining a concrete thickness that will be sufficient, in conjunction with other sources of resistance, to resist the buoyant force at the bottom of the seal.

Hydrostatic pressure head is defined from the bottom of tremie seal up to the ordinary water surface elevation. **BDM 7.03.06B**

Consult the geotechnical engineer for the hydrostatic pressure head and the related information.

Hydrostatic pressure head to the bottom of the footing (from the geotechnical engineer) $H_{water} := 10ft$

Unit weight of water $\gamma_{water} := 62.4\frac{lb}{ft^3}$

Unit weight of tremie concrete $\gamma_{tremie} := 140\frac{lb}{ft^3}$

Consider a 1-ft wide strip of the tremie seal.

Assume that the cofferdam sheet piling is located at 18 in. outside the footing outline. **BDM 7.03.04**

Note: It is recommended to evaluate the possibility of battered piles hitting the cofferdam sheet piles during the pile driving operation. Based on the pile layout design, evaluate the space requirements and increase the distance between the footing outline and the cofferdam sheet piles. Use the revised dimensions to calculate the length of the tremie seal. This example uses the following dimensions:

Length of tremie seal $l_{tremie} := B_{footing} + 3ft = 14\,ft$

Tremie seal thickness (assumed) $h_{tremie} := 4ft$

Weight of the tremie seal $W_{tremie} := \gamma_{tremie}\cdot l_{tremie}\cdot 1ft\cdot h_{tremie} = 7.84\cdot kip$

Bending stress check

While checking the flexural stress in the tremie seal, assume that the 1-ft wide tremie seal strip is simply supported at both ends by the cofferdam.

Section modulus of the 1-ft wide strip $S_{tremie} := \frac{1}{6}\cdot h_{tremie}^2 = 2.667\cdot ft^2$

Max. bending moment in the tremie seal

$$M_{tremie} := \frac{1}{8}\left[\gamma_{water}\cdot\left(H_{water} + h_{tremie}\right) - \gamma_{tremie}\cdot h_{tremie}\right]\cdot l_{tremie}^2 = 7.683\cdot\frac{kip\cdot ft}{ft}$$

Max. tensile stress in the tremie seal $F_c := \frac{M_{tremie}}{S_{tremie}} = 20.008\cdot psi$

Check if the max. tensile stress is less than 30 psi $Check := if\left(F_c < 30psi, "OK", "Not OK"\right) = "OK"$

Friction Check

Bond strength between tremie and cofferdam $p_{CoffBond} := 5psi$ **BDM 7.03.06A**

Bond force between tremie and cofferdam $P_{CoffBond} := p_{CoffBond} 2 \cdot 1ft \cdot h_{tremie} = 5.76 \cdot kip$

From Section 4.6, the selected pile section HP14X73

Note: For H piles, surface area of a pile is defined using a rectangular shape for a conservative design (i.e. 2 times the sum of the flange width and the section depth).

Flange width $b_f = 14.585 \cdot in$

Section depth $d_{pile} = 13.61 \cdot in$

Bond strength between tremie and piles $p_{PileBond} := 10psi$ **BDM 7.03.06A**

Bond force between tremie and piles when a 1-ft wide strip is considered

$$P_{PileBond} := p_{PileBond} \cdot \left(\frac{N_{front}}{L_{footing}} + \frac{N_{back}}{L_{footing}} \right) \cdot 2 \cdot 1ft \cdot \left(b_f + d_{pile} \right) \cdot h_{tremie} = 9.468 \cdot kip$$

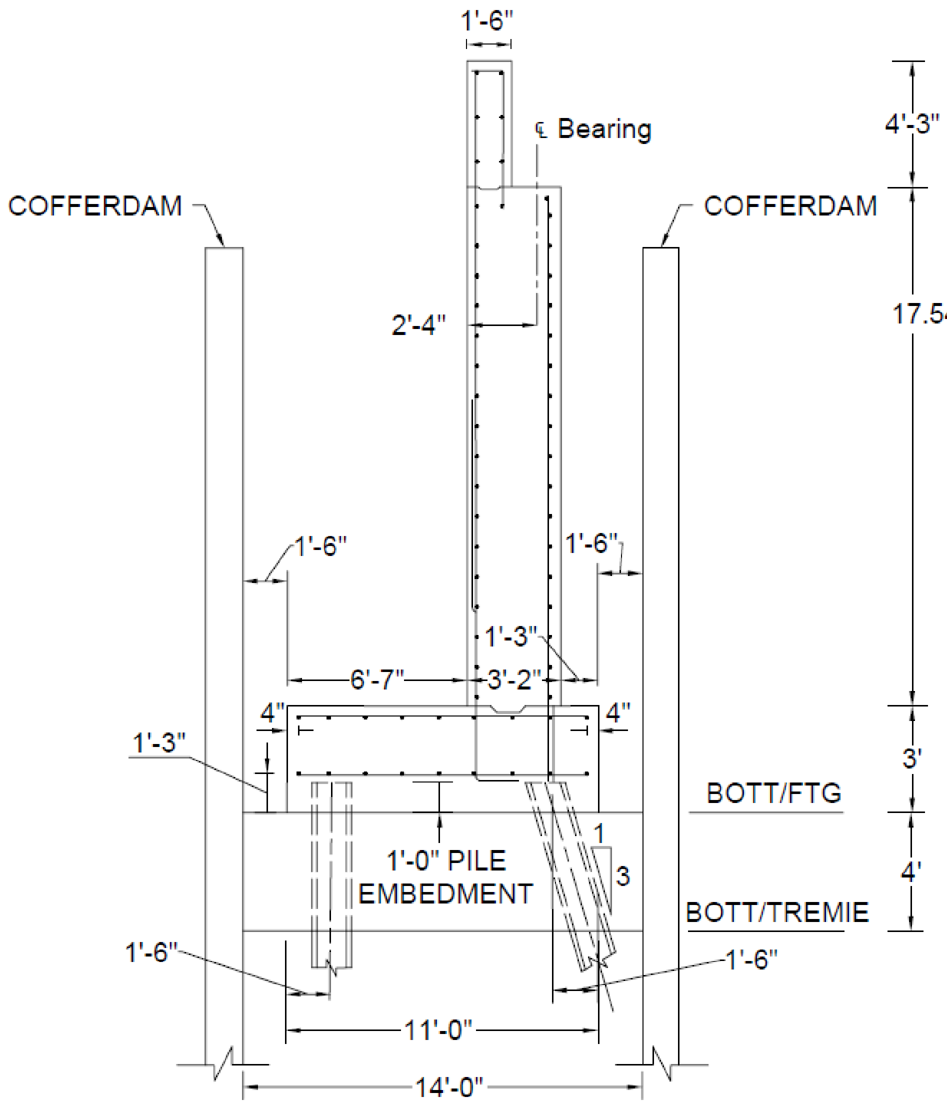
Total uplift resistance (capacity) $P_{resist} := W_{tremie} + P_{CoffBond} + P_{PileBond}$

$P_{resist} = 23.068 \cdot kip$ **BDM 7.03.06A**

Buoyant force (demand) $P_{buoy} := \gamma_{water} \cdot 1ft \, l_{tremie} \cdot \left(H_{water} + h_{tremie} \right) = 12.23 \cdot kip$

Check if the total resistance force > the buoyant force $Check := if\left(P_{resist} > P_{buoy}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

The figure in the following page show a sketch of the pile foundation with the designed tremie seal.



Note: Refer to MDOT Bridge Design Guides for additional bars, laps, embedment, keyway dimensions, and drainage details. They are not shown in this drawing for clarity of main reinforcement.

Based on the pile layout design, evaluate the required space for driving battered piles and possible conflicts with the cofferdam sheet piles. Increase the distance between the footing outline and the cofferdam sheet piles when needed, and recheck the tremie design.