DESIGN OF A MULTI-COLUMN PIER AND FOUNDATIONS

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Section 5 Multi-Column Pier with Spread Footing

Step 5.1 Preliminary Dimensions

Description

This section illustrates the design of a multi-column pier supported by a spread footing for an interstate freeway bridge. The design is implemented in accordance with the Michigan Department of Transportation (MDOT) policies published as of 09/30/2022. The requirements of the 9th Edition of the AASHTO LRFD Bridge Design Specification; as modified and supplemented by the Bridge Design Manual (BDM), Bridge Design Guides (BDG), and 2020 Standard Specifications for Construction (SSFC); are followed. Certain material and design parameters are selected to be in compliance with MDOT practice reflected in the Bridge Design System (BDS), the MDOT legacy software.

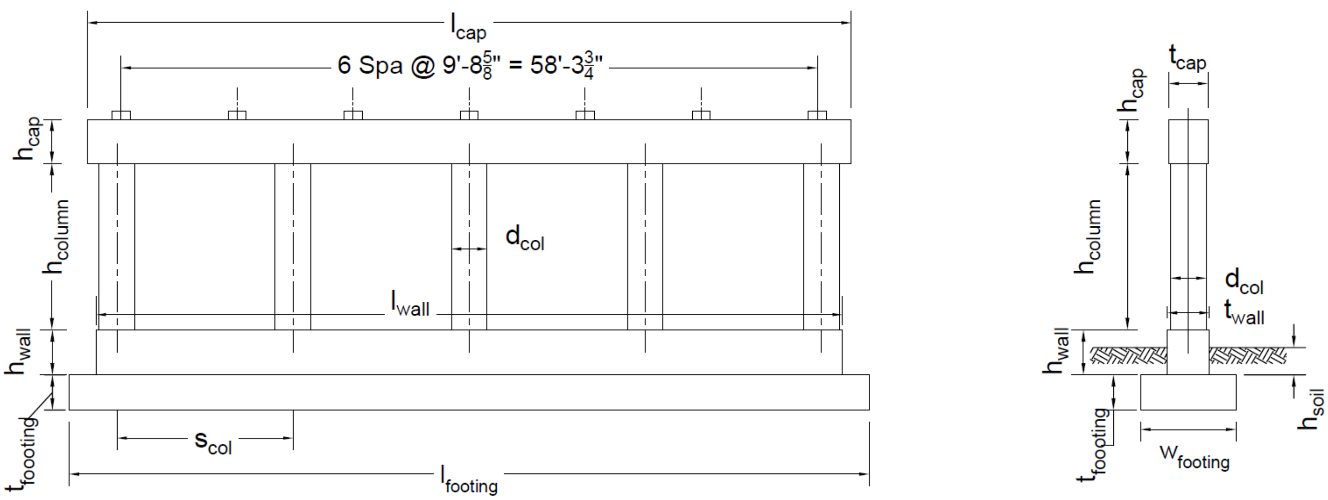
The pier is designed for the superstructure described in the *Two-Span Continuous Bridge Steel Plate Girder Design Example* developed by Attanayake et al. (2021). Refer to Section 2 of the *Design of Highway Bridge Abutments and Foundations Example* developed by Attanayake and Hu (2023) for the design criteria, bridge information, material properties, and soil types and properties.

These examples are available at https://mdotjboss.state.mi.us/SpecProv/trainingmaterials.htm#2108560.

The preliminary dimensions are selected based on site-specific conditions, highway agency standards, and past experience.

Construction joints should be provided when the pier cap is longer than 25 ft. **BDM 7.03.03.C.3**
A 1-in. open joint may be required to control temperature moment in long piers with short columns.

The following figure shows the pier geometry and dimensional variables:



The preliminary dimensions selected for this example are given below.

Pier cap length $l_{cap} := 64ft$

Pier cap height $h_{cap} := 3.5ft$

The pier cap is to be approximately 3 in. wider than the column diameter and should provide 4.5 in. minimum clearance between the edge of masonry plate (or elastomeric pad) and the face of the cap. **BDM 7.03.03.C.1**

Pier cap width $t_{cap} := 2.75ft$

Pier cap overhang length $Overhang_{cap} := 2.5ft$

In general, 2.5 ft diameter columns should be used with beams less than 42 in., unless loading conditions or bearing areas dictate larger columns. The beam depth of the bridge is 33 in. **BDM 7.03.03.B.1**

Column diameter $d_{col} := 2.5ft$

Columns should be spaced far enough apart so as to be appealing to the eye; if the beam spacing is far enough apart, a column may be placed under each bearing. Use a maximum column diameter (or width) to column spacing ratio of 1:8. **BDM 7.03.03.B.4**

Column spacing $S_{col} := 14.75ft$ **BDM 7.03.03.B.4**

Maximum column diameter to column spacing ratio

$$Check := if\left(\frac{d_{col}}{S_{col}} > \frac{1}{8}, \text{"OK"}, \text{"Reduce Column Spacing"}\right) = \text{"OK"}$$

Column height from the bottom of the cap to the top of the base wall $h_{col} := 12ft$

No. of columns $n_{col} := 5$

Piers that are within the clear zone or in a median where barriers are required should have base walls. The base wall is to be 3 in. wider than the column to prevent vehicle snagging and should extend a minimum of 3.5 ft above the ground line. **BDM 7.03.03.D**

BDG 5.22.01 and 5.24.01 provide base wall details for piers located adjacent to roadways and railway tracks, respectively. **BDG 5.22.01** **BDG 5.24.01**

Note: According to BDG 5.22.01, the base wall should be 6 in. wider than the column, which contradicts with the BDM 7.03.03.D requirements. This example does, in fact, use a base wall that is 6 in. wider than the column diameter.

Note: For piers located adjacent to roadways, the minimum height of the base wall shall be 5 ft with a minimum of 3.5 ft extended above the ground or shoulder. For piers located adjacent to railway tracks, the minimum height of the base wall shall be 11ft with a minimum of 6 ft extended above the top of the rail head. Designers shall refer to BDG 5.24 for additional requirements for bridge piers located near the railway tracks.

This example demonstrates the design of a pier located adjacent to a roadway and supported on a spread footing.

Difference between base wall thickness and column diameter $diff_{wall} := 6in$

Base wall thickness $t_{wall} := d_{col} + diff_{wall} = 3\,ft$

Base wall length $l_{wall} := \left(n_{col} - 1\right)\cdot S_{col} + d_{col} + 6in = 62\,ft$

Base wall height $h_{wall} := 6ft$

Footing length $l_{footing} := 69ft$

The minimum footing thickness for bridge piers located adjacent to roadways and railway tracks is 2.5 ft. **BDG 5.22.01** **BDG 5.24.01**

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

Footing width $w_{footing} := 10ft$

Depth of soil located on top of the footing $h_{soil} := 2ft$

Note: The depth from the ground level to the bottom of the footing needs to be maintained at a minimum of 4 ft for frost depth. Typically, a one-foot deep soil profile is maintained with normal grading when the pier is at a median. The depth of the soil may change to 2 to 3 ft based on the pavement profile when the pier is closer to the pavement.

Concrete cover requirements for reinforcing steel

Unless otherwise shown on the plans, the minimum concrete clear cover for reinforcement shall satisfy the following requirements: **BDM 8.02.N**

Concrete cast against earth: 3 in.
For all other cases unless shown on plans: 2 in.

The following concrete cover is used since it is greater than the required minimum. **BDG 5.22.01**

Cover for the base wall side face $Cover_{wall} := 3.75in$

Cover for the base wall top $Cover_{walltop} := 3in$

Cover for the footing $Cover_{ft} := 4in$

Since the concrete cover requirements for pier caps and columns are not provided in the BDM and BDG, the following dimensions are taken from the MDOT Sample Bridge Plans.

Cover for the bend cap $Cover_{cap} := 3.5in$

Cover for the columns $Cover_{col} := 4in$

Step 5.2 Application of Dead Load

Description

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

Dead Load Girder Reactions

The superstructure dead load reactions per bearing are taken from the *Steel Plate Girder Design Example*. All the beam seats are assumed to be at the same elevation.

When calculating superstructure loads on the substructure, 75% of the barrier dead load should be applied with the fascia beam load. The remaining 25% of the barrier load should be applied with the first interior girder load. **BDM 7.01.04.J**

Note: The exterior and interior girder shear values presented in the *Steel Plate Girder Design Example* (Table 12 and 13) were calculated by equally distributing the barrier loads to all the girders. Therefore, the girder reactions over the pier due to barrier loads need to be recalculated as shown below.

Exterior Girders **Table 12 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DCEx\_noBarrier} := 161.4kip$$

Reaction due to 75% of the barrier weight (DB) on the exterior girder $R_{DCEx\_barrier} := 44kip$

Total exterior girder reaction due to DC $R_{DCEx} := R_{DCEx\_noBarrier} + R_{DCEx\_barrier} = 205.4 \cdot kip$

Reaction due to the weight of the future wearing surface (DW) $R_{DWEx} := 26.6kip$

First Interior Girder **Table 13 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DC1stIn\_noBarrier} := 190.4kip$$

Reaction due to 25% of the barrier weight (DB) on the first interior girder $R_{DC1stIn\_barrier} := 14.5kip$

Total first interior girder reaction due to DC $R_{DC1stIn} := R_{DC1stIn\_noBarrier} + R_{DC1stIn\_barrier} = 204.9 \cdot kip$

Weight of the future wearing surface (DW) $R_{DWIn} := 26.4kip$

Other Interior Girders **Table 13 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DCIn} := 190.4kip$$

Dead Load Calculation

Dead load of superstructure

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

$$DC_{Sup} := 2 \cdot R_{DCEx} + 2 \cdot R_{DC1stIn} + \left(N_{beams} - 4\right) \cdot R_{DCIn}$$

$$DC_{Sup} = 1.392 \times 10^{3} \cdot kip$$

Weight of the future wearing surface (DW) $DW_{Sup} := 2 \cdot R_{DWEx} + \left(N_{beams} - 2\right) \cdot R_{DWIn} = 185.2 \cdot kip$

Pier cap weight $DC_{cap} := l_{cap} \cdot h_{cap} \cdot t_{cap} \cdot W_c = 92.4 \cdot kip$

Pier column weight $DC_{column} := n_{col} \frac{1}{4} \pi \, d_{col}^{2} \cdot h_{col} \cdot W_c = 44.179 \cdot kip$

Base wall weight $DC_{wall} := l_{wall} \cdot t_{wall} \cdot h_{wall} \cdot W_c = 167.4 \cdot kip$

Footing weight $DC_{footing} := w_{footing} \cdot l_{footing} \cdot t_{footing} \cdot W_c = 310.5 \cdot kip$

Step 5.3 Application of Live Load

Description

This step describes the application of the live load on the pier.

Girder Reactions under a Single Lane Load

MDOT uses a modified version of the HL-93 loading in the AASHTO LRFD Bridge Design Specifications. The combination of a single design truck load, a single 60-kip load (axle load), or a two design truck load for continuous spans, and a design lane load are multiplied by a factor of 1.2 to designate the design loading as HL-93 Mod.

Factor for HL-93 Mod	$f_{HL93Mod} := 1.2$	**BDM 7.01.04-A**
Dynamic load allowance	$IM := 0.33$	**LRFD Table 3.6.2.1-1**

Even though several software programs are available for designers to calculate the maximum live load effects by developing 3D bridge models and simulating live load positions along and across the lanes, this example demonstrates a commonly used, easy to implement, approach for the same purpose. The process includes the following steps:

1. Develop a single line girder model representing girder cross-sections, effective deck cross-section, composite and noncomposite segments of the girder, and boundary conditions.
2. Apply relevant truck and lane loads as independent loads to calculate the maximum reaction per lane at the girder support over the pier. For example, refer to Table A-2 and A-4 in the *Steel Plate Girder Design Example* for loads and the format of results.
3. Multiply selected support reactions with applicable factors. For example, the support reaction due to truck load is multiplied by the impact factor. When the support reactions are due to the truck pair for continuous spans, both reactions due to truck and lane loads are multiplied by a factor of 0.9 to account for the 10% reduction specified in the AASHTO LRFD Bridge Design Specifications.
4. Calculate an equivalent pair of wheel loads, P_{wheel}, that will result in the same support reactions developed by the truck load on a single lane.
5. Calculate an equivalent 10-ft long line load, W_{lane}, that will result in the same support reactions developed by the lane load applied on a single lane.
6. Apply these P_{wheel} and W_{lane} loads on the bridge deck to generate girder end reactions that will ultimately result in the maximum force effects in the pier cap, columns, base wall, and footing.

Since the load distribution factors in the LRFD Specifications are not used in this process to calculate girder end reactions, a multiple presence factor is applied to the truck and lane loads depending on the number of design lanes considered in the analysis.

$$MPF(lane) := \begin{cases} 1.2 & \text{if } lane = 1 \\ 1.0 & \text{if } lane = 2 \\ 0.85 & \text{if } lane = 3 \\ 0.65 & \text{otherwise} \end{cases}$$

LRFD Table 3.6.1.1.2-1

The *Steel Plate Girder Design Example* presents unfactored girder reactions for truck and lane loads. The following three live load cases are used in the *Steel Plate Girder Design Example* to determine the design forces and moments:

Case 1: design truck + design lane, **LRFD 3.6.1.3.1**
Case 2: a single 60-kip axle load + design lane, and
Case 3: 90% of two design trucks spaced a minimum of 50-ft apart + 90% of design lane.

Case 1 is the governing case for girder reactions over the pier. Case 1 reactions given in Appendix A of the *Steel Plate Girder Design Example* on a per lane basis do not include the factors for HL-93 Mod and the dynamic load allowance.

Table A-2 and A-4 in the *Steel Plate Girder Design Example* present the exterior and interior girder reactions per lane. As shown in Table A-2 and A-4, the exterior girder reactions are slightly greater than the interior girder reactions. For this design, exterior girder reactions are used to calculate P_{wheel} and W_{lane} loads.

Since the load Case 1 (i.e. the design truck + lane load combination) produces the maximum reactions over the pier, the following loads are selected.

Maximum reaction at the girder supports over the pier due to the design truck load

$$V_{Truck} := 67.6kip$$

Table A-2 of the Steel Plate Girder Design Example

Maximum reaction at the girder supports over the pier due to the design lane load

$$V_{Lane} := 40.5kip$$

Table A-2 of the Steel Plate Girder Design Example

The unfactored concentrated load representing the girder reaction per wheel line

$$P_{wheel} := \frac{V_{Truck}}{2} \cdot f_{HL93Mod} \cdot (1 + IM) = 53.945 \cdot kip$$

Next, the unfactored uniformly distributed load representing girder reactions over the pier due to the design lane load is calculated. This load is transversely distributed over a 10 ft wide strip. The dynamic load allowance is not applied to this load.

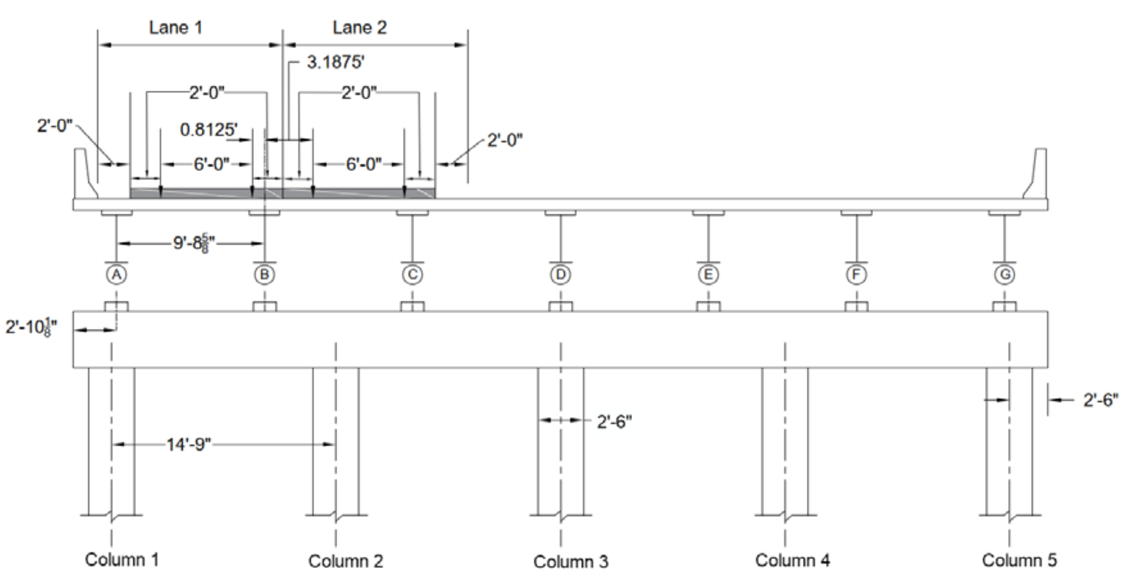
LRFD 3.6.2.1

$$W_{lane} := \frac{V_{Lane} \cdot f_{HL93Mod}}{10ft} = 4.86 \cdot \frac{kip}{ft}$$

Critical Live Load Positions and Girder Reactions for Pier Cap and Column Design

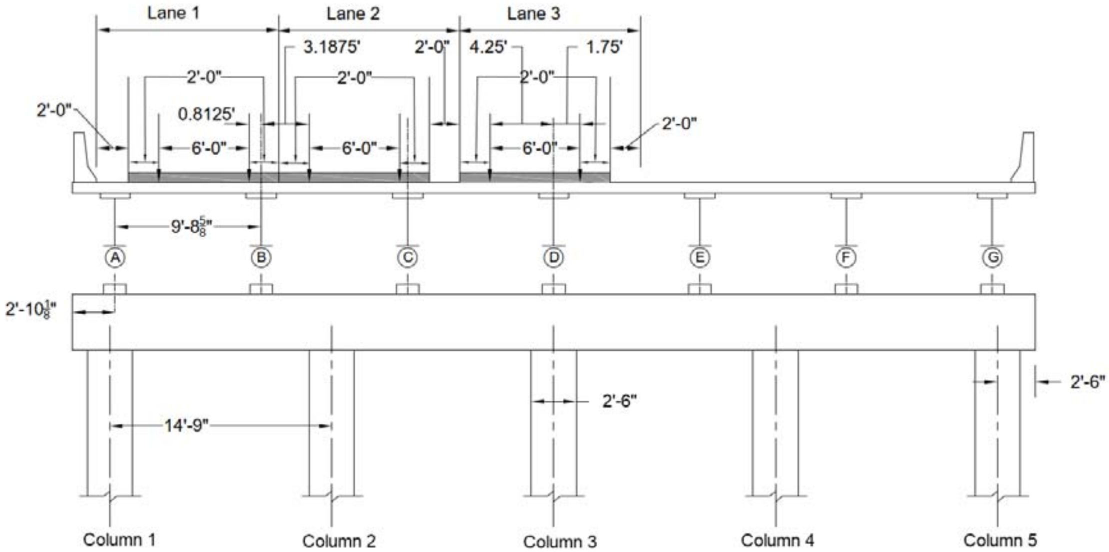
This superstructure can accommodate a maximum of five (5) 12-ft wide design lanes. Therefore, the maximum live load effects on the pier cap, columns, base wall, and footing are determined by considering a combination of one, two, three, four, or five loaded lanes. Since the width of the lane load is 10 ft and the axle is 6 ft, these loads are placed across the 12-ft wide lane to develop the girder end reactions that ultimately result in the maximum force effects on the pier cap, columns, base wall, and footing.

The following figures illustrate the loaded lane positions to determine the maximum positive and negative moments and shear forces in the pier cap. As shown in figure (a), two lanes are loaded to maximize girder B reactions in order to develop the maximum positive moment in the 1st bay of the pier cap. The positions of the loads are decided based on the influence lines provided in Appendix 5.A.



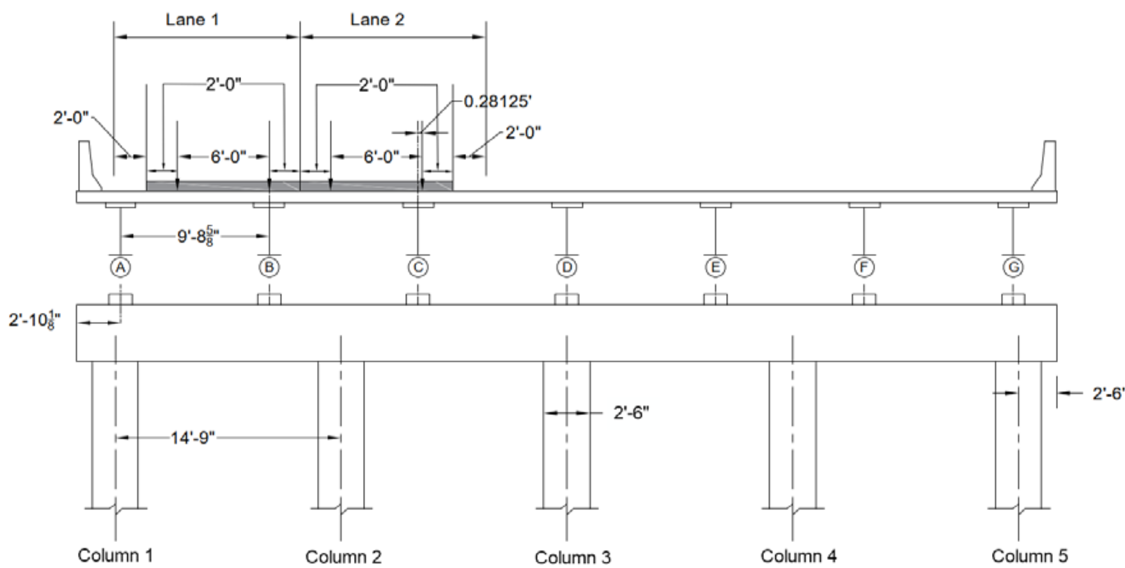
(a) Lane and Load Positions to Develop the Maximum Positive Moment in the Pier Cap

Figure (b) shows the position of three lanes to maximize girder B and C reactions in order to develop the maximum negative pier cap moment over Column 2. Lane loads are placed closer to the edge of the lane while the wheel loads are placed two feet from the edge of the lanes as per the AASHTO LRFD guidelines.



(b) Lane and Load Positions to Develop the Maximum Negative Moment in the Pier Cap

Figure (c) shows the live load position to generate the maximum shear in the 1st bay of the pier cap.



(c) Lane and Load Positions to Develop the Maximum Shear in the Pier Cap

The next step is to compute the reactions due to the above loads at each of the seven bearing locations. The reactions are calculated by assuming that the deck is pinned at the interior girder locations but continuous over the exterior girders.

Girder End Reactions to Develop the Maximum Positive Pier Cap Moment

Girder end reactions are calculated considering the load configuration shown in Figure (a).

Number of lanes loaded $\quad lanes := 2$

$$R_{A.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{0.8125ft}{S}+\frac{6.8125ft}{S}\right)+\frac{W_{lane}\cdot(8.8125ft)^2}{2S}\right] = 61.741\cdot kip$$

$$R_{B.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{2.90625ft}{S}+\frac{8.90625ft}{S}+\frac{6.53125ft}{S}+\frac{0.53125ft}{S}\right)\ldots\right.$$
$$\left.+\frac{W_{lane}\cdot 8.8125ft\cdot(S-0.5\cdot 8.8125ft)}{S}+W_{lane}\cdot\frac{S}{2}\right] = 151.795\cdot kip$$

$$R_{C.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{3.1875ft}{S}+\frac{9.1875ft}{S}\right)+W_{lane}\cdot\frac{S}{2}\ldots\right.$$
$$\left.+\frac{W_{lane}\cdot 1.46875ft\cdot(S-0.5\cdot 1.46875ft)}{S}\right] = 98.904\cdot kip$$

$$R_{D.LL} := MPF(lanes)\cdot\left[W_{lane}\cdot\frac{(1.46875ft)^2}{2\cdot S}\right] = 0.539\cdot kip \qquad R_{E.LL} := 0kip \quad R_{F.LL} := 0kip \quad R_{G.LL} := 0kip$$

Girder End Reactions to Develop the Maximum Negative Pier Cap Moment

Girder end reactions are calculated considering the load configuration shown in Figure (b).

Number of lanes loaded $\quad lanes := 3$

$$R_{A.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{0.8125ft}{S}+\frac{6.8125ft}{S}\right)+\frac{W_{lane}\cdot(8.8125ft)^2}{2S}\right] = 52.48\cdot kip$$

$$R_{B.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{2.90625ft}{S}+\frac{8.90625ft}{S}+\frac{6.53125ft}{S}+\frac{0.53125ft}{S}\right)\ldots\right.$$
$$\left.+\frac{W_{lane}\cdot 8.8125ft\cdot(S-0.5\cdot 8.8125ft)}{S}+W_{lane}\cdot\frac{S}{2}\right] = 129.026\cdot kip$$

$$R_{C.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{3.1875ft}{S}+\frac{9.1875ft}{S}+\frac{4.25ft}{S}\right)\ldots\right.$$
$$\left.+\left[W_{lane}\cdot\frac{S}{2}+\frac{W_{lane}\cdot 1.46875ft\cdot(S-0.5\cdot 1.46875ft)}{S}+\frac{W_{lane}\cdot(6.25ft)^2}{2S}\right]\right] = 112.422\cdot kip$$

$$R_{D.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{S-4.25ft}{S}+\frac{S-1.75ft}{S}\right)+\frac{W_{lane}\cdot 6.25ft\cdot(S-0.5\cdot 6.25ft)}{S}\ldots\right.$$
$$\left.+\frac{W_{lane}\cdot(1.4675ft)^2}{2S}+W_{lane}\cdot\frac{3.75ft\cdot(S-0.5\cdot 3.75ft)}{S}\right] = 93.875\cdot kip$$

$$R_{E.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{1.75ft}{S}\right) + W_{lane}\cdot\frac{(3.75ft)^2}{2S}\right] = 11.245\cdot kip \quad R_{F.LL} := 0kip \qquad R_{G.LL} := 0kip$$

Girder End Reactions to Develop the Maximum Shear in the Pier Cap

Girder end reactions are calculated considering the load configuration shown in Figure (c).

Number of lanes loaded $lanes := 2$

$$R_{A.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{6ft}{S}\right) + \frac{W_{lane}\cdot(8ft)^2}{2S}\right] = 49.306\cdot kip$$

$$R_{B.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(1 + \frac{3.71875ft}{S} + \frac{5.71875ft}{S}\right) \ldots \atop + \frac{W_{lane}\cdot 8ft\cdot(S - 0.5\cdot 8ft)}{S} + W_{lane}\cdot\frac{S}{2}\right] = 152.823\cdot kip$$

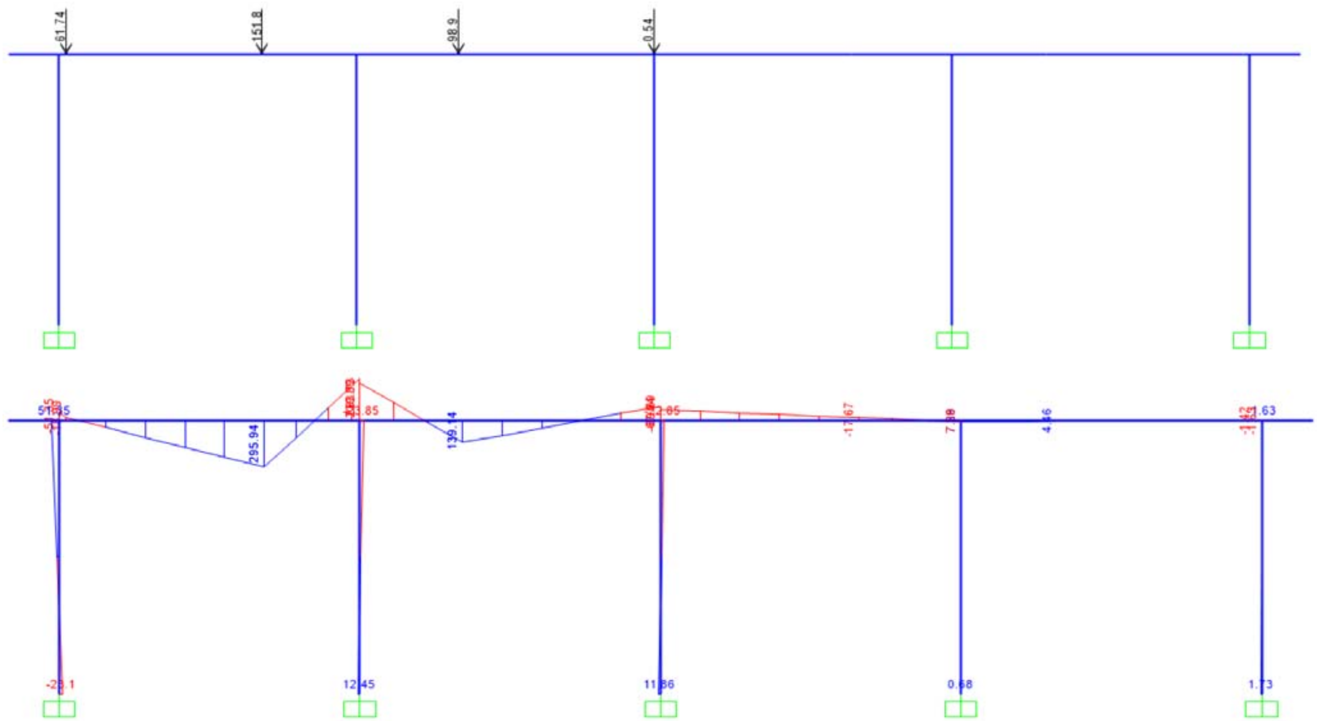
$$R_{C.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{4ft}{S} + \frac{S - 0.28125ft}{S}\right) + W_{lane}\cdot\frac{S}{2} \ldots \atop + \frac{W_{lane}\cdot 2.28125ft\cdot(S - 0.5\cdot 2.28125ft)}{S}\right] = 107.988\cdot kip$$

$$R_{D.LL} := MPF(lanes)\cdot\left[P_{wheel}\cdot\left(\frac{0.28125ft}{S}\right) + W_{lane}\cdot\frac{(2.28125ft)^2}{2\cdot S}\right] = 2.862\cdot kip$$

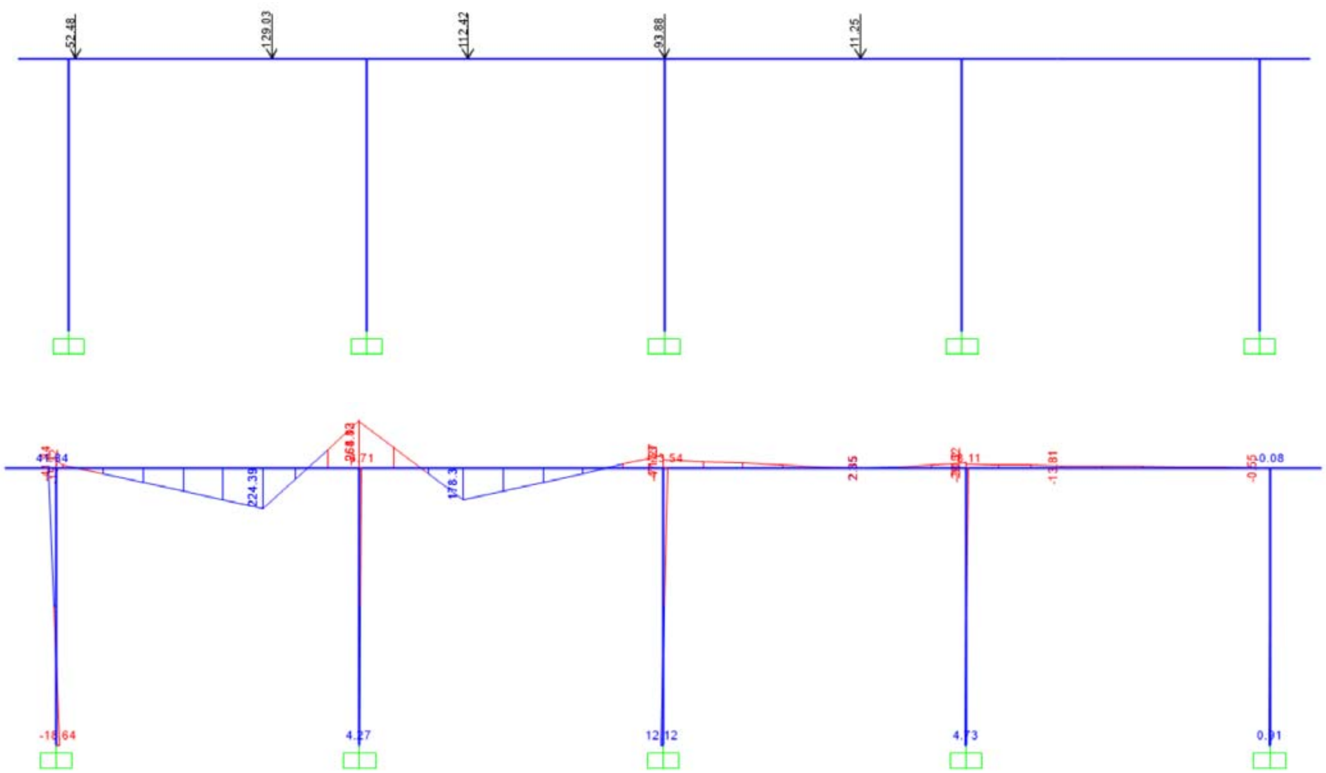
$R_{E.LL} := 0kip$ $\qquad R_{F.LL} := 0kip$ $\qquad R_{G.LL} := 0kip$

These girder reactions are then applied on the pier cap to calculate the maximum moments and shear forces. The following figures show pier models with the applied girder reactions and the corresponding moment or shear diagrams.

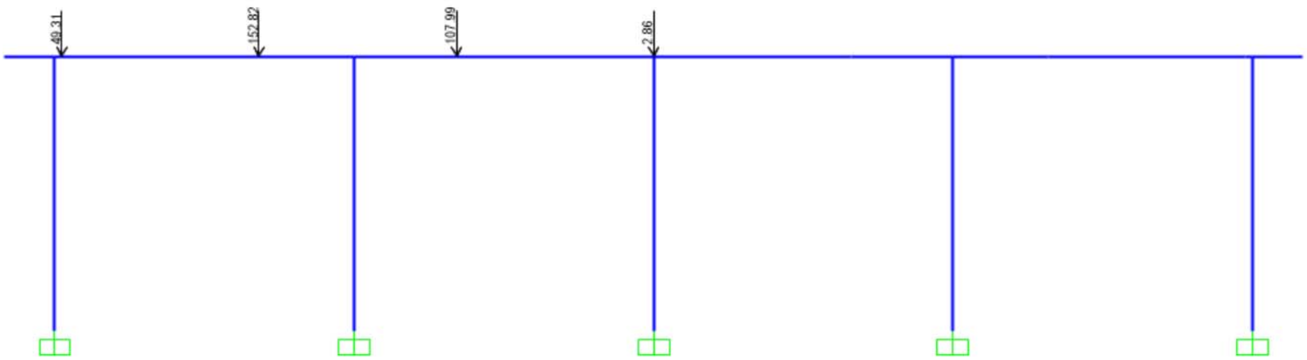
Maximum Positive Live Load Moment

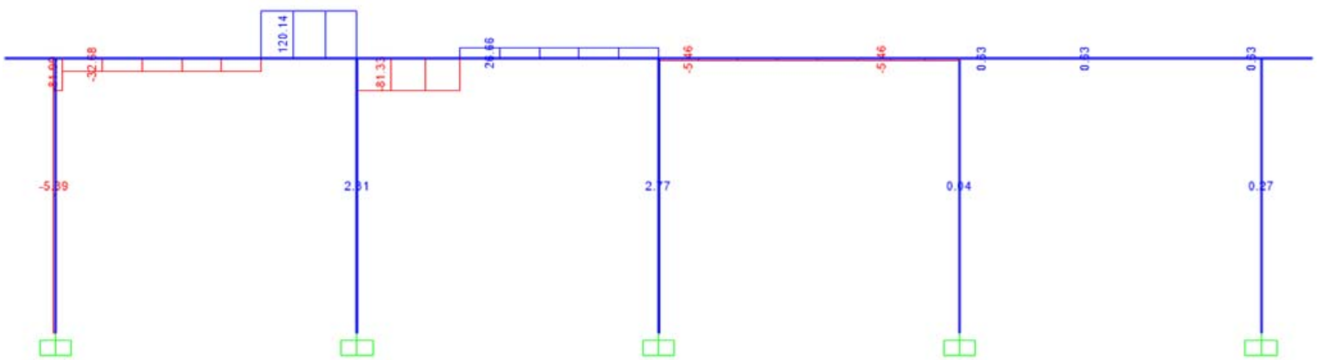


Maximum Negative Live Load Moment



Maximum Live Load Shear





Critical Live Load Positions and Girder Reactions for Footing Design

Because of the damping effect of soil, only the static effects of the design truck or tandem are considered for the design of footings. Hence, the unfactored concentrated load per wheel line is calculated by excluding the dynamic load allowance as shown below. **LRFD 3.6.2.1**

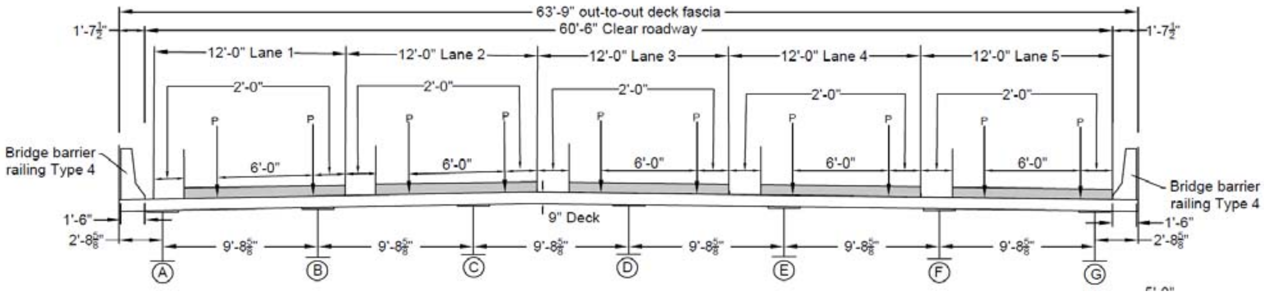
The unfactored concentrated load per wheel line used in the design of the footing

$$P_{wheel\_ft} := \frac{V_{Truck}}{2} \cdot f_{HL93Mod} = 40.56 \cdot kip$$

The dynamic load allowance is not applied to the design lane load. Therefore, the unfactored uniformly distributed load (representing girder reactions over the pier due to the design lane load) for the footing design is

$$W_{lane} = 4.86 \cdot \frac{kip}{ft}$$

The design live load should be placed to generate the maximum soil bearing pressure. The following figure illustrates the position of wheel and lane loads to generate the greatest eccentricity and loads to maximize the soil bearing pressure.



Next, the girder reactions are calculated considering each loaded lane.

Only Lane 5 is loaded

$$R_{G5\_ft} := \frac{P_{wheel\_ft} \cdot (8.8125ft + 2.2125ft) + W_{lane} \cdot 10ft \cdot 5.8125ft}{S} = 75.078 \cdot kip$$

$$R_{F5\_ft} := P_{wheel\_ft} \cdot 2 + W_{lane} \cdot 10ft - R_{G5\_ft} = 54.642 \cdot kip$$

$R_{A5\_ft} := 0$ $\quad R_{B5\_ft} := 0$ $\quad R_{C5\_ft} := 0$ $\quad R_{D5\_ft} := 0$ $\quad R_{E5\_ft} := 0$

Only Lane 4 is loaded

$$R_{F4\_ft} := \frac{P_{wheel\_ft} \cdot (6.53125ft + 0.53125ft) + W_{lane} \cdot 8.53125ft \cdot (0.5 \cdot 8.53125ft)}{S} = 47.672 \cdot kip$$

$$R_{D4\_ft} := \frac{W_{lane} \cdot 1.46875ft \cdot (0.5 \cdot 1.46875ft)}{S} = 0.539 \cdot kip$$

$$R_{E4\_ft} := P_{wheel\_ft} \cdot 2 + W_{lane} \cdot 10ft - R_{F4\_ft} - R_{D4\_ft} = 81.508 \cdot kip$$

$R_{A4\_ft} := 0$ $\quad R_{B4\_ft} := 0$ $\quad R_{C4\_ft} := 0$ $\quad R_{G4\_ft} := 0$

Only Lane 3 is loaded

$$R_{E3\_ft} := \frac{P_{wheel\_ft} \cdot (4.25ft) + W_{lane} \cdot 6.25ft \cdot (0.5 \cdot 6.25ft)}{S} = 27.504 \cdot kip$$

$$R_{C3\_ft} := \frac{P_{wheel\_ft} \cdot (1.75ft) + W_{lane} \cdot 3.75ft \cdot (0.5 \cdot 3.75ft)}{S} = 10.819 \cdot kip$$

$$R_{D3\_ft} := P_{wheel\_ft} \cdot 2 + W_{lane} \cdot 10ft - R_{E3\_ft} - R_{C3\_ft} = 91.397 \cdot kip$$

$R_{A3\_ft} := 0$ $\quad R_{B3\_ft} := 0$ $\quad R_{F3\_ft} := 0$ $\quad R_{G3\_ft} := 0$

Only Lane 2 is loaded:

$$R_{D2\_ft} := \frac{P_{wheel\_ft} \cdot (1.96875ft) + W_{lane} \cdot 3.96875ft \cdot (0.5 \cdot 3.96875ft)}{S} = 12.155 \cdot kip$$

$$R_{B2\_ft} := \frac{P_{wheel\_ft} \cdot (4.03125ft) + W_{lane} \cdot 6.03125ft \cdot (0.5 \cdot 6.03125ft)}{S} = 25.919 \cdot kip$$

$$R_{C2\_ft} := P_{wheel\_ft} \cdot 2 + W_{lane} \cdot 10ft - R_{B2\_ft} - R_{D2\_ft} = 91.646 \cdot kip$$

$R_{A2\_ft} := 0$ $\quad R_{E2\_ft} := 0$ $\quad R_{F2\_ft} := 0$ $\quad R_{G2\_ft} := 0$

Only Lane 1 is loaded

$$R_{A1\_ft} := \frac{P_{wheel\_ft} \cdot (0.3125ft + 6.3125ft) + W_{lane} \cdot 8.3125ft \cdot (0.5 \cdot 8.3125ft)}{S} = 44.925 \cdot kip$$

$$R_{C1\_ft} := \frac{W_{lane} \cdot 1.6875ft \cdot (0.5 \cdot 1.6875ft)}{S} = 0.712 \cdot kip$$

$$R_{B1\_ft} := P_{wheel\_ft} \cdot 2 + W_{lane} \cdot 10ft - R_{A1\_ft} - R_{C1\_ft} = 84.083 \cdot kip$$

$R_{D1\_ft} := 0$ $\quad R_{E1\_ft} := 0$ $\quad R_{F1\_ft} := 0$ $\quad R_{G1\_ft} := 0$

All 5 lanes are loaded

$$R_{AFt\_5L} := \left(R_{A1\_ft} + R_{A2\_ft} + R_{A3\_ft} + R_{A4\_ft} + R_{A5\_ft}\right) \cdot MPF(5) = 29.201 \cdot kip$$

$$R_{BFt\_5L} := \left(R_{B1\_ft} + R_{B2\_ft} + R_{B3\_ft} + R_{B4\_ft} + R_{B5\_ft}\right) \cdot MPF(5) = 71.501 \cdot kip$$

$$R_{CFt\_5L} := \left(R_{C1\_ft} + R_{C2\_ft} + R_{C3\_ft} + R_{C4\_ft} + R_{C5\_ft}\right) \cdot MPF(5) = 67.066 \cdot kip$$

$$R_{DFt\_5L} := \left(R_{D1\_ft} + R_{D2\_ft} + R_{D3\_ft} + R_{D4\_ft} + R_{D5\_ft}\right) \cdot MPF(5) = 67.659 \cdot kip$$

$$R_{EFt\_5L} := \left(R_{E1\_ft} + R_{E2\_ft} + R_{E3\_ft} + R_{E4\_ft} + R_{E5\_ft}\right) \cdot MPF(5) = 70.858 \cdot kip$$

$$R_{FFt\_5L} := \left(R_{F1\_ft} + R_{F2\_ft} + R_{F3\_ft} + R_{F4\_ft} + R_{F5\_ft}\right) \cdot MPF(5) = 66.505 \cdot kip$$

$$R_{GFt\_5L} := \left(R_{G1\_ft} + R_{G2\_ft} + R_{G3\_ft} + R_{G4\_ft} + R_{G5\_ft}\right) \cdot MPF(5) = 48.801 \cdot kip$$

The total unfactored live load when all 5 lanes are loaded

$$R_{LLFooting} := R_{AFt\_5L} + R_{BFt\_5L} + R_{CFt\_5L} + R_{DFt\_5L} + R_{EFt\_5L} + R_{FFt\_5L} + R_{GFt\_5L} = 421.59 \cdot kip$$

Step 5.4 Application of Other Loads

Description

This step describes the application of braking force, wind load, temperature load, earth load, and vehicle collision load. Other loads, such as ice load and centrifugal force are not applicable for this example. For illustrative purposes, the calculation of ice load and centrifugal force are presented in Appendix 5.B and 5.C.

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Braking Force

Since the abutments have expansion bearings, the fixed bearings at the pier resist the braking force in the longitudinal direction.

The braking force (BR) shall be taken as the greater of: **LRFD 3.6.4**

- 25% of the axle weight of the design truck / tandem
- 5% of the design truck / tandem weight plus lane load

The braking force is applied on all design lanes assuming that the bridge carries traffic in one direction.

Braking force per lane due to 25% of the axle weight of the design truck / tandem

$$BR_1 := 25\% \cdot (32kip + 32kip + 8kip) = 18 \cdot kip$$

Braking force per lane due to 5% of the design truck / tandem weight plus lane load

$$BR_2 := 5\% \cdot \left(72kip + 0.64\frac{kip}{ft} \cdot 2L_{span}\right) = 10 \cdot kip$$

Note: The MDOT practice, as reflected in the BDS, is to take only 5% of the design truck plus the lane load as the breaking force. In addition, the HL-93 modification factor is not included in the braking force calculation. This example follows the MDOT practice.

Braking force selected for the design $BRK := BR_2 = 10 \cdot kip$

Next, calculate the braking force considering 1 to 5 loaded lanes.

Braking force due to 1 loaded lane $BRK_{1L} := BRK \cdot MPF(1) = 12 \cdot kip$

Braking force due to 2 loaded lanes $BRK_{2L} := 2BRK \cdot MPF(2) = 20 \cdot kip$

Braking force due to 3 loaded lanes $BRK_{3L} := 3BRK \cdot MPF(3) = 25.5 \cdot kip$

Braking force due to 4 loaded lanes $BRK_{4L} := 4BRK \cdot MPF(4) = 26 \cdot kip$

Braking force due to 5 loaded lanes $BRK_{5L} := 5BRK \cdot MPF(5) = 32.5 \cdot kip$

The braking force is assumed to be equally shared by the bearings at the pier.

The braking force shall be assumed to act horizontally at a distance of 6 ft above the roadway surface. **LRFD 3.6.4**

Note: The MDOT practice is to apply the horizontal component of the breaking force at the bearings. The impact of the eccentricity of the load with respect to the bearing elevation is not considered.

Wind Load

Since the expansion bearings are located over the abutments, the fixed bearings at the pier resist the longitudinal component of the wind load acting on the superstructure.

Wind Load on Superstructure

LRFD 3.8.1.1, 3.8.1.2

To calculate the wind load acting on the superstructure, the total depth from the top of the barrier to the bottom of the girder is considered. Then, the wind exposure area is calculated by multiplying the tributary length for a specific direction and the superstructure depth. Finally, the wind load is calculated by multiplying the wind pressure and the wind exposure area.

Since the expansion bearings at the abutment are restrained in the transverse direction, the tributary length for the transverse direction wind load on the pier with fixed bearings is equal to one-half of each adjacent span. Because of the expansion bearings at the abutments, the entire bridge length is selected as the tributary length for the longitudinal direction.

Total depth of the superstructure $D_{total} := h_{Railing} + t_{Deck} + t_{Haunch} + d_{Girder} = 7.083\ \text{ft}$

Tributary length for the transverse wind load on superstructure $L_{WindT} := L_{span} = 100\ \text{ft}$ (one half of each adjacent span)

Tributary length for the longitudinal superstructure wind load $L_{WindL} := 2L_{span} = 200\ \text{ft}$ (entire bridge span)

Effective area for the transverse wind load on superstructure $A_{WSuperT} := D_{total} \cdot L_{WindT} = 708.333\ \text{ft}^2$

Effective area for the longitudinal wind load on superstructure $A_{WSuperL} := D_{total} \cdot L_{WindL} = 1.417 \times 10^3\ \text{ft}^2$

Basic wind speed (mph) for Strength III load combination $V_{wStrIII} := 115$ **LRFD Figure 3.8.1.1.2-1**

Basic wind speed (mph) for Strength V load combination $V_{wStrV} := 80$ **LRFD Table 3.8.1.1.2-1**

Basic wind speed (mph) for Service I load combination $V_{wSerI} := 70$ **LRFD Table 3.8.1.1.2-1**

Gust effect factor $Gust := 1$ **LRFD Table 3.8.1.2.1-1, no sound barrier**

Drag coefficient, superstructure $C_{DSup} := 1.3$ **LRFD Table 3.8.1.2.1-2**

Superstructure height (ft) when the height is less than 33 ft $Z := 33$ **LRFD 3.8.1.2.1**

Wind exposure category B

Pressure exposure and elevation coefficient for Strength III and Service IV load combinations

$$K_{ZSup} := \frac{\left(2.5 \cdot \ln\left(\frac{Z}{0.9832}\right) + 6.87\right)^2}{345.6} = 0.709$$

LRFD Eq. 3.8.1.2.1-2

The wind pressure acting on the superstructure is calculated for different load combinations. **LRFD Eq. 3.8.1.2.1-1**

Wind pressure on the superstructure (ksf), Strength III $P_{ZSup.StrIII} := 2.56 \cdot 10^{-6} \cdot K_{ZSup} \cdot V_{wStrIII}^2 \cdot Gust \cdot C_{DSup} = 0.031$

Wind pressure on the superstructure (ksf), Strength V $P_{ZSup.StrV} := 2.56 \cdot 10^{-6} \cdot V_{wStrV}^2 \cdot Gust \cdot C_{DSup} = 0.021$

Wind pressure on the superstructure (ksf), Service I $P_{ZSup.SerI} := 2.56 \cdot 10^{-6} \cdot V_{wSerI}^2 \cdot Gust \cdot C_{DSup} = 0.016$

The superstructure wind load acting on the pier depends on the wind attack angle which is measured from a line perpendicular to the longitudinal axis of the bridge. **LRFD 3.8.1.2.2**

Since the span length and height of this girder bridge are less than 150 ft and 33 ft respectively, the following wind load components are used: **LRFD 3.8.1.2.3a**

- Transverse: 100 percent of the wind load calculated based on the wind direction perpendicular to the longitudinal axis of the bridge
- Longitudinal: 25 percent of the transverse load.

Only the pier has fixed bearings. Therefore, the longitudinal component of the wind load on the superstructure is equally shared by the bearings at the pier.

Wind load at each bearing due to the transverse wind loads on the superstructure, Strength III

$$WS_{TStrIII} := \frac{P_{ZSup.StrIII} \cdot ksf \cdot A_{WSuperT}}{N_{beams}} = 3.158 \cdot kip$$

Wind load at each bearing due to the transverse wind loads on the superstructure, Strength V

$$WS_{TStrV} := \frac{P_{ZSup.StrV} \cdot ksf \cdot A_{WSuperT}}{N_{beams}} = 2.155 \cdot kip$$

Wind load at each bearing due to the transverse wind loads on the superstructure, Service I

$$WS_{TSerI} := \frac{P_{ZSup.SerI} \cdot ksf \cdot A_{WSuperT}}{N_{beams}} = 1.65 \cdot kip$$

Wind load at each bearing due to the longitudinal wind loads on the superstructure, Strength III

$$WS_{LStrIII} := WS_{TStrIII} \cdot \frac{A_{WSuperL}}{A_{WSuperT}} \cdot 0.25 = 1.579 \cdot kip$$

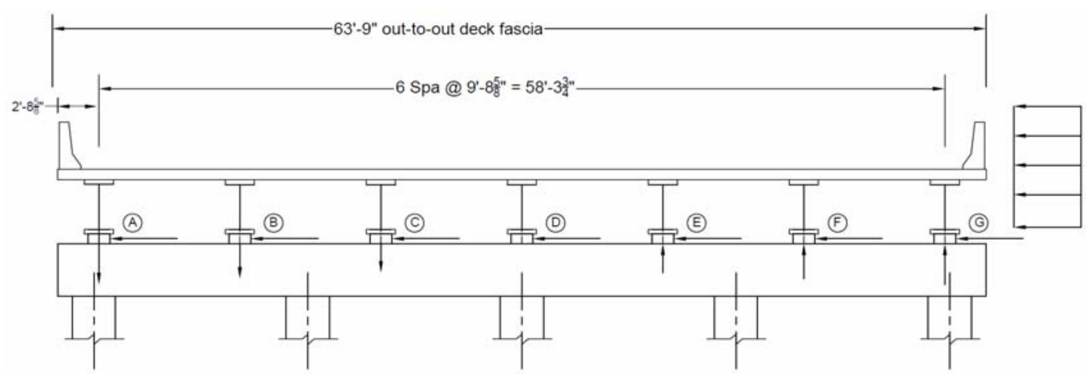
Wind load at each bearing due to the longitudinal wind loads on the superstructure, Strength V

$$WS_{LStrV} := WS_{TStrV} \cdot \frac{A_{WSuperL}}{A_{WSuperT}} \cdot 0.25 = 1.078 \cdot kip$$

Wind load at each bearing due to the longitudinal wind loads on the superstructure, Service I

$$WS_{LSerI} := WS_{TSerI} \cdot \frac{A_{WSuperL}}{A_{WSuperT}} \cdot 0.25 = 0.825 \cdot kip$$

The transverse load acting on the superstructure also applies a moment to the pier cap. This moment acts about the transverse centerline of the pier cap and induces vertical loads at the bearings, as illustrated in the following figure.



The following calculations show the moments about the longitudinal axis of the bridge due to the transverse wind loads on the superstructure:

Strength III

$$M_{TStrIII} := P_{ZSup.StrIII} \cdot ksf \cdot A_{WSuperT} \cdot \frac{D_{total}}{2} = 78.285 \cdot kip \cdot ft$$

Strength V

$$M_{TStrV} := P_{ZSup.StrV} \cdot ksf \cdot A_{WSuperT} \cdot \frac{D_{total}}{2} = 53.433 \cdot kip \cdot ft$$

Service I

$$M_{TSerI} := P_{ZSup.SerI} \cdot ksf \cdot A_{WSuperT} \cdot \frac{D_{total}}{2} = 40.91 \cdot kip \cdot ft$$

Moment of inertia for the girder group

$$I_{girders} := 2 \cdot (3S)^2 + 2(2S)^2 + 2 \cdot S^2 = 2.645 \times 10^3 \, ft^2$$

Vertical forces at bearings A and G, Strength III $R_{WS\_AGStrIII} := \frac{M_{TStrIII} \cdot (3S)}{I_{girders}} = 0.863 \cdot kip$

Vertical forces at bearings A and G, Strength V $R_{WS\_AGStrV} := \frac{M_{TStrV} \cdot (3S)}{I_{girders}} = 0.589 \cdot kip$

Vertical forces at bearings A and G, Service I $R_{WS\_AGSerI} := \frac{M_{TSerI} \cdot (3S)}{I_{girders}} = 0.451 \cdot kip$

Vertical forces at bearings B and F, Strength III $R_{WS\_BFStrIII} := \frac{M_{TStrIII} \cdot (2S)}{I_{girders}} = 0.575 \cdot kip$

Vertical forces at bearings B and F, Strength V $R_{WS\_BFStrV} := \frac{M_{TStrV} \cdot (2S)}{I_{girders}} = 0.393 \cdot kip$

Vertical forces at bearings B and F, Service I $R_{WS\_BFSerI} := \frac{M_{TSerI} \cdot (2S)}{I_{girders}} = 0.301 \cdot kip$

Vertical forces at bearings C and E, Strength III $R_{WS\_CEStrIII} := \frac{M_{TStrIII} \cdot (S)}{I_{girders}} = 0.288 \cdot kip$

Vertical forces at bearings C and E, Strength V $R_{WS\_CEStrV} := \frac{M_{TStrV} \cdot (S)}{I_{girders}} = 0.196 \cdot kip$

Vertical forces at bearings C and E, Service I $R_{WS\_CESerI} := \frac{M_{TSerI} \cdot (S)}{I_{girders}} = 0.15 \cdot kip$

Note: The MDOT practice is to equally distribute the horizontal component of the transverse wind load to the bearings and neglect the effect of eccentricity. The above calculation is for illustrative purposes only. The vertical forces induced at the bearings by the eccentric transverse wind load are not considered in the design.

Vertical Wind Load

The vertical upward wind load is calculated as 0.02 ksf times the width of the deck for the Strength III load combination. This line load is applied at the windward quarter of the deck width. **LRFD 3.8.2**

Note: Since the MDOT practice is not to consider the vertical wind load, it is excluded from the analysis and design presented in this example .

Wind Load on Substructure

Drag coefficient, substructure $C_{DSub} := 1.6$ **LRFD Table 3.8.1.2.1-2**

The wind pressure acting on the substructure is calculated for different load combinations. **LRFD Eq. 3.8.1.2.1-1**

Wind pressure on the substructure (ksf), Strength III $P_{ZSub.StrIII} := 2.56 \cdot 10^{-6} \cdot K_{ZSup} \cdot V_{wStrIII}^{2} \cdot Gust \cdot C_{DSub} = 0.038$

Wind pressure on the substructure (ksf), Strength V $P_{ZSub.StrV} := 2.56 \cdot 10^{-6} \cdot V_{wStrV}^{2} \cdot Gust \cdot C_{DSub} = 0.026$

Wind pressure on the substructure (ksf), Service I

$$P_{ZSub.SerI} := 2.56 \cdot 10^{-6} \cdot V_{wSerI}^{2} \cdot Gust \cdot C_{DSub} = 0.02$$

For simplicity, apply the same pressure along the transverse and longitudinal directions. The transverse and longitudinal wind forces calculated from these wind pressures acting on the corresponding exposed areas are to be applied simultaneously. These loads shall also act simultaneously with the superstructure wind loads.

LRFD 3.8.1.2.3b

The wind loads acting on the pier are calculated as line loads; they are applied at the center line of the pier cap and columns. A computer model is used to calculate the moments and forces developed in the pier under these wind loads.

Parallel to the Transverse Direction of the Bridge

Wind loads acting on the pier cap ($P_{TPierCap}$) are applied as concentrated loads at the mid-depth of the pier cap.

Wind load acting on the pier cap, Strength III

$$P_{TPierCap.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot h_{cap} \cdot t_{cap} = 0.37 \cdot kip$$

Wind load acting on the pier cap, Strength V

$$P_{TPierCap.StrV} := P_{ZSub.StrV} \cdot ksf \cdot h_{cap} \cdot t_{cap} = 0.252 \cdot kip$$

Wind load acting on the pier cap, Service I

$$P_{TPierCap.SerI} := P_{ZSub.SerI} \cdot ksf \cdot h_{cap} \cdot t_{cap} = 0.193 \cdot kip$$

Wind loads acting on the columns (W_{TCol}) are only applied on the exterior column in the windward direction as uniformly distributed loads.

Wind load acting on the column, Strength III

$$W_{TCol.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot d_{col} = 0.096 \cdot \frac{kip}{ft}$$

Wind load acting on the column, Strength V

$$W_{TCol.StrV} := P_{ZSub.StrV} \cdot ksf \cdot d_{col} = 0.066 \cdot \frac{kip}{ft}$$

Wind load acting on the column, Service I

$$W_{TCol.SerI} := P_{ZSub.SerI} \cdot ksf \cdot d_{col} = 0.05 \cdot \frac{kip}{ft}$$

Wind loads acting on the base wall (P_{TWall}) are applied as concentrated loads at the mid-depth of the above-ground portion of the wall.

Wind load acting on the base wall, Strength III

$$P_{TWall.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot t_{wall} = 0.461 \cdot kip$$

Wind load acting on the base wall, Strength V

$$P_{TWall.StrV} := P_{ZSub.StrV} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot t_{wall} = 0.315 \cdot kip$$

Wind load acting on the base wall, Service I

$$P_{TWall.SerI} := P_{ZSub.SerI} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot t_{wall} = 0.241 \cdot kip$$

Parallel to the Longitudinal Direction of the Bridge

Wind loads acting on the pier cap ($W_{LPierCap}$) are applied at the mid-depth of the cap as uniformly distributed loads.

Wind load acting on the pier cap, Strength III

$$W_{LPierCap.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot h_{cap} = 0.134 \cdot \frac{kip}{ft}$$

Wind load acting on the pier cap, Strength V

$$W_{LPierCap.StrV} := P_{ZSub.StrV} \cdot ksf \cdot h_{cap} = 0.092 \cdot \frac{kip}{ft}$$

Wind load acting on the pier cap, Service I

$$W_{LPierCap.SerI} := P_{ZSub.SerI} \cdot ksf \cdot h_{cap} = 0.07 \cdot \frac{kip}{ft}$$

Wind loads acting on the columns (W_{LCol}) are applied along the vertical centerline of the columns as uniformly distributed loads.

Wind load acting on the column, Strength III $$W_{LCol.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot d_{col} = 0.096 \cdot \frac{kip}{ft}$$

Wind load acting on the column, Strength V $$W_{LCol.StrV} := P_{ZSub.StrV} \cdot ksf \cdot d_{col} = 0.066 \cdot \frac{kip}{ft}$$

Wind load acting on the column, Service I $$W_{LCol.SerI} := P_{ZSub.SerI} \cdot ksf \cdot d_{col} = 0.05 \cdot \frac{kip}{ft}$$

Wind loads acting on the base wall (P_{LWall}) are applied as concentrated loads at the mid-depth of the above-ground portion of the wall.

Wind load acting on the base wall, Strength III $$P_{LWall.StrIII} := P_{ZSub.StrIII} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot l_{wall} = 9.525 \cdot kip$$

Wind load acting on the base wall, Strength V $$P_{LWall.StrV} := P_{ZSub.StrV} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot l_{wall} = 6.501 \cdot kip$$

Wind load acting on the base wall, Service I $$P_{LWall.SerI} := P_{ZSub.SerI} \cdot ksf \cdot \left(h_{wall} - h_{soil}\right) \cdot l_{wall} = 4.977 \cdot kip$$

Wind Load on Live Load

Since the individual span length and height of this girder bridge are less than 150 ft and 33 ft respectively, the following wind load components acting on the live load are used: **LRFD 3.8.1.3**

- 0.10 klf, transverse
- 0.04 klf, longitudinal.

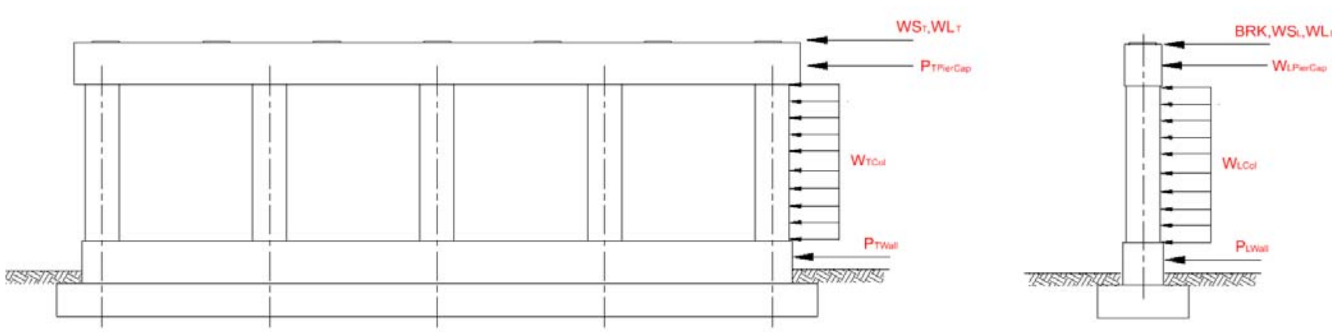
The transverse and longitudinal components of the load acting on each bearing are:

$$WL_{TBearing} := \frac{0.1 \frac{kip}{ft} \cdot L_{WindT}}{N_{beams}} = 1.429 \cdot kip \qquad WL_{LBearing} := \frac{0.04 \frac{kip}{ft} \cdot L_{WindL}}{N_{beams}} = 1.143 \cdot kip$$

The wind load on live load acts at 6 ft above the roadway. **LRFD 3.8.1.3**

Note: The MDOT procedure does not consider the eccentricity of the wind load acting on live load. Only the horizontal force is distributed to the bearings.

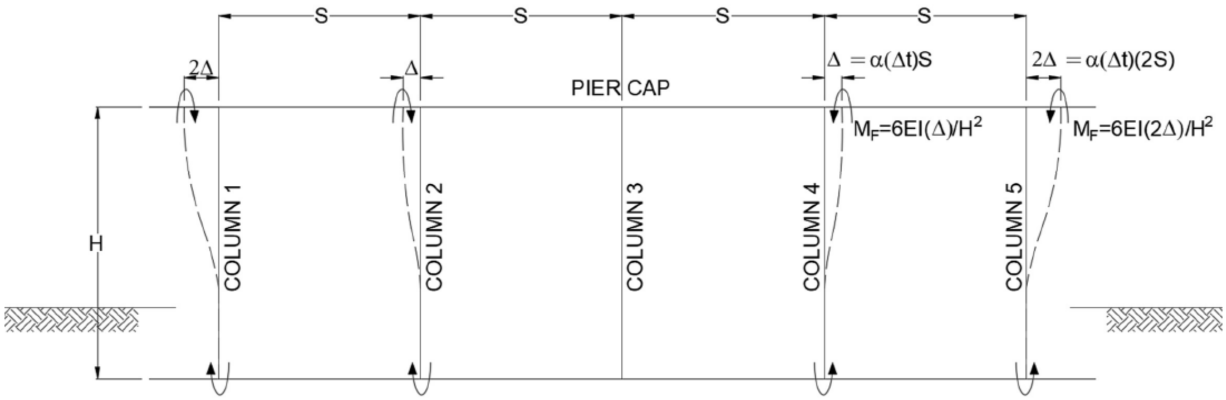
The following figure shows the braking force and the wind loads applied on the pier in the transverse and longitudinal direction of the bridge:



Temperature Load

Since this bridge has two equal spans and expansion bearings over the abutments, the center of movement in the longitudinal direction is located at the pier. Therefore, the bearing pads at the pier do not deform when the superstructure deforms due to change in temperature. As a result, the pier is not subjected to transverse forces.

The pier cap expansion and contraction due to change in temperature from the reference value at the time of construction induces moments in the columns as shown below.



Thermal expansion coefficient of concrete (/°F) $\quad \alpha := 6 \cdot 10^{-6}$ **BDM 7.01.07 cold climate temperature range**

Use a 45° F drop and a 35° F rise from the temperature at the time of construction.

Contraction and expansion temperature ranges $\quad T_{contraction} := 45 \qquad T_{expansion} := 35$

The contraction of a pier cap segment located between two columns due to change in temperature

$$\Delta_{TContr} := \alpha \cdot S_{col} \cdot T_{contraction} = 0.048 \cdot in$$

The expansion of a pier cap segment located between two columns due to change in temperature

$$\Delta_{TExp} := \alpha \cdot S_{col} \cdot T_{expansion} = 0.037 \cdot in$$

Moment of inertia of the column cross-section

$$I_{col} := \frac{1}{64} \cdot \pi \, d_{col}^{4} = 1.917\, ft^{4}$$

Moment acting on columns 1 and 5 due to contraction

$$M_{TCol15Contr} := \frac{6E_c \cdot I_{col} \cdot 2\Delta_{TContr}}{h_{col}^{2}} = 332.226 \cdot kip \cdot ft$$

Moment acting on columns 1 and 5 due to expansion

$$M_{TCol15Exp} := \frac{6E_c \cdot I_{col} \cdot 2\Delta_{TExp}}{h_{col}^{2}} = 258.398 \cdot kip \cdot ft$$

Moment acting on columns 2 and 4 due to contraction

$$M_{TCol24Contr} := \frac{6E_c \cdot I_{col} \cdot \Delta_{TContr}}{h_{col}^{2}} = 166.113 \cdot kip \cdot ft$$

Moment acting on columns 2 and 4 due to expansion

$$M_{TCol24Exp} := \frac{6E_c \cdot I_{col} \cdot \Delta_{TExp}}{h_{col}^{2}} = 129.199 \cdot kip \cdot ft$$

Since the pier cap deformation due to contraction is greater, the moments induced in columns due to contraction is considered for further analysis and design.

Vertical Earth Load

Vertical earth load on the footing

$$EV_{Ft} := \gamma_s \cdot \left(l_{footing} \cdot w_{footing} - l_{wall} \cdot t_{wall}\right) \cdot h_{soil} = 120.96 \cdot kip$$

Vehicle Collision Load

The draft language for incorporating AASHTO LRFD vehicle collision force is being reviewed by the bridge committee. Once approved, the AASHTO LRFD vehicle collision force shall be accounted for in the design of all new bridges, bridge replacements, and pier replacements.

MDOT's preference is to locate the pier outside of the clear zone as defined in Section 7.01.11 of the MDOT Road Design Manual. After the draft language is approved, the updated BDM will describe the the preference for accounting for the vehicle collision force when the pier cannot be located outside of the clear zone.

The pier design described in this example does not consider the vehicle collision force assuming that the pier is located outside of the clear zone defined in Section 7.01.11 of the MDOT Road Design Manual.

Step 5.5 Combined Load Effects

Description

This step presents the procedure of combining all load effects and calculating the total factored forces and moments acting on the pier cap, columns, base wall, and footing.

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Strength I, Strength III, Strength V, and Service I limit states are considered for the analysis and design of the pier. **LRFD 3.4.1**

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

Strength III = 1.25DC + 1.5DW + 1.5EH + 1.35EV + 1.0WS + 0.5TU

Strength V = 1.25DC + 1.5DW + 1.35LL + 1.35BR + 1.0WS + 1.0WL + 1.5EH + 1.35EV + 1.35LS + 0.5TU

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

BR	= vehicular braking force
DC	= dead load of structural components and nonstructural attachments
DW	= dead load of future wearing surface and utilities
EH	= horizontal earth pressure load
EV	= vertical pressure from the earth fill
LL	= vehicular live load
LS	= live load surcharge
WL	= wind on live load
WS	= wind load on structure
TU	= force effect due to uniform temperature

Limit states that are not shown either do not control or are not applicable.

Note: These load combinations should include the maximum and minimum load factors; only the maximum factors are shown for clarity.

Forces and Moments at the Pier Cap

The pier cap moments and forces were calculated using a structural analysis software. The pier is modeled as a plane frame with fixed supports. The superstructure dead and live loads, braking force, and the wind load acting on the superstructure and vehicles are modeled as point loads at the bearing locations. The wind load acting on the substructure is modeled as point loads and distributed loads on the pier cap and columns, respectively.

By examination of the load effects, the Strength I limit state is identified as the controlling limit state for pier cap design. The critical load effects on the pier cap include the maximum positive moment, the maximum negative moment, and the maximum shear. Service I limit state results are also needed for the design. The critical load effects under Strength I and Service I limit states are listed below.

<u>Strength I Limit State</u>

Factored maximum positive moment $M_{uPStrI} := 1032.4\,kip \cdot ft$

Note: As per **LRFD C5.6.3.2.1**, this example uses the negative moment at the face of the column. For the circular column in this example, the face of the column is assumed to be located at the face of an equivalent square area concentric with the circular column.

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Equivalent column width $a_{col} := \sqrt{\frac{\pi \cdot d_{col}^2}{4}} = 2.216\,ft$

Factored maximum negative moment at the face of the column $M_{uNStrI} := 680.7\,kip \cdot ft$

Factored maximum shear at the face of the column $V_{uPCapStrI} := 475.5\,kip$

Note: The shear design in Step 5.6 uses the shear at the critical section as specified in the LRFD specifications.

Service I Limit State

Factored maximum positive moment $M_{uPSerI} := 709.0 kip \cdot ft$

Factored maximum negative moment at the face of the column $M_{uNSerI} := 473.2 kip \cdot ft$

Factored maximum shear at the face of the column $V_{uPCapSerI} := 331.0 kip$

Forces and Moments at the Pier Columns

The model used to calculate pier cap moments and forces is used to calculate the column loads. The critical section of the column is located at the top of the base wall. By examination of the load effects, Strength I or Strength V are identified as the controlling limit states for the column design. The critical load effects under Strength I and Strength V limit states are listed below.

Strength I Limit State

Axial load at the critical section $P_{u\_StrI} := 862.6 kip$

Moment about the longitudinal axis of the bridge $M_{ut\_StrI} := 159.2 kip \cdot ft$

Moment about the transverse axis of the bridge $M_{ul\_StrI} := 147.34 kip \cdot ft$

Shear parallel to the transverse axis of the bridge $V_{ut\_StrI} := 20.6 kip$

Shear parallel to the longitudinal axis of the bridge $V_{ul\_StrI} := 10.1 kip$

Strength V Limit State

Axial load at the critical section $P_{u\_StrV} := 780.8 kip$

Moment about the longitudinal axis of the bridge $M_{ut\_StrV} := 165.8 kip \cdot ft$

Moment about the transverse axis of the bridge $M_{ul\_StrV} := 176.0 kip \cdot ft$

Shear parallel to the transverse axis of the bridge $V_{ut\_StrV} := 23.3 kip$

Shear parallel to the longitudinal axis of the bridge $V_{ul\_StrV} := 11.7 kip$

Forces and Moments at the Base Wall

The live load on all five lanes develops the critical load effects at the base of the wall.

The braking force, the wind load on superstructure, and the wind load acting on the live load are applied at the bearings. The moment arm of these forces to the bottom of the base wall is

$$Arm_{wall} := h_{cap} + h_{col} + h_{wall} = 21.5 \ ft$$

Strength I

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

All Lanes Loaded

Factored shear force parallel to the transverse axis of the wall $V_{WallStrI} := 1.75 \cdot BRK_{5L} = 56.875 \cdot kip$

Factored moment about the longitudinal axis of the wall

$$M_{WallStrI} := 1.75 \cdot BRK_{5L} \cdot Arm_{wall} = 1.223 \times 10^{3} \cdot kip \cdot ft$$

Strength III

Strength III = 1.25DC + 1.5DW + 1.5EH + 1.35EV + 1.0WS + 0.5TU

Factored shear force parallel to the transverse axis of the wall

$$V_{WallStrIII} := N_{beams} \cdot WS_{LStrIII} + W_{LPierCap.StrIII} \cdot l_{cap} + n_{col} \cdot W_{LCol.StrIII} \cdot h_{col} + P_{LWall.StrIII}$$

$$V_{WallStrIII} = 34.941 \cdot kip$$

Factored moment about the longitudinal axis of the wall

$$M_{WallStrIII} := N_{beams} \cdot WS_{LStrIII} \cdot Arm_{wall} + W_{LPierCap.StrIII} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall}\right) \dots$$
$$+ n_{col} \cdot W_{LCol.StrIII} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall}\right) + P_{LWall.StrIII} \cdot \left(\frac{h_{wall} + h_{soil}}{2}\right)$$

$$M_{WallStrIII} = 514.765 \cdot kip \cdot ft$$

Strength V

Strength V = 1.25DC + 1.5DW + 1.35LL + 1.35BR + 1.0WS + 1.0WL + 1.5EH + 1.35EV + 1.35 LS + 0.5TU

<u>All Lanes Loaded</u>

Factored shear force parallel to the transverse axis of the wall

$$V_{WallStrV} := 1.35 \cdot BRK_{5L} + N_{beams} \cdot \left(WS_{LStrV} + WL_{LBearing}\right) \dots$$
$$+ W_{LPierCap.StrV} \cdot l_{cap} + n_{col} \cdot W_{LCol.StrV} \cdot h_{col} + P_{LWall.StrV}$$

$$V_{WallStrV} = 75.724 \cdot kip$$

Factored moment about the longitudinal axis of the wall

$$M_{WallStrV} := 1.35 \cdot BRK_{5L} \cdot Arm_{wall} + N_{beams} \cdot \left(WS_{LStrV} \cdot Arm_{wall} + WL_{LBearing} \cdot Arm_{wall}\right) \dots$$
$$+ W_{LPierCap.StrV} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall}\right) \dots$$
$$+ n_{col} \cdot W_{LCol.StrV} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall}\right) + P_{LWall.StrV} \cdot \left(\frac{h_{wall} + h_{soil}}{2}\right)$$

$$M_{WallStrV} = 1.467 \times 10^{3} \cdot kip \cdot ft$$

Controlling shear and moment applied on the base wall

$$V_{Wall} := \max\left(V_{WallStrI}, V_{WallStrIII}, V_{WallStrV}\right) = 75.724 \cdot kip$$

$$M_{Wall} := \max\left(M_{WallStrI}, M_{WallStrIII}, M_{WallStrV}\right) = 1.467 \times 10^{3} \cdot kip \cdot ft$$

Service I

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

Factored shear force parallel to the transverse axis of the wall

$$V_{WallSerI} := BRK_{5L} + N_{beams} \cdot \left(WS_{LSerI} + WL_{LBearing}\right) \ldots$$
$$+ W_{LPierCap.SerI} \cdot l_{cap} + n_{col} \cdot W_{LCol.SerI} \cdot h_{col} + P_{LWall.SerI}$$

$$V_{WallSerI} = 58.759 \cdot kip$$

Factored moment about the longitudinal axis of the wall

$$M_{WallSerI} := BRK_{5L} \cdot Arm_{wall} + N_{beams} \cdot \left(WS_{LSerI} \cdot Arm_{wall} + WL_{TBearing} \cdot Arm_{wall}\right) \ldots$$
$$+ W_{LPierCap.SerI} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall}\right) \ldots$$
$$+ n_{col} \cdot W_{LCol.StrV} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall}\right) + P_{LWall.SerI} \cdot \left(\frac{h_{wall} + h_{soil}}{2}\right)$$

$$M_{WallSerI} = 1.194 \times 10^{3} \cdot kip \cdot ft$$

Forces and Moments at the Pier Footing

The bearing pressure distribution depends on the rigidity of the footing and the soil type and condition. The pier footings are usually rigid, and the assumption q = (P/A) +/- (Mc/I) is valid. For an accurate calculation of bearing pressure distribution, the footing may be analyzed as a beam on an elastic foundation.

The live load on all five lanes develop the critical load effects for footing design.

Moment arm of Girder A and G reactions to the center of footing $\quad Arm_{AG} := 3S = 29.156\,ft$

Moment arm of Girder B and F reactions to the center of footing $\quad Arm_{BF} := 2S = 19.438\,ft$

Moment arm of Girder C and E reactions to the center of footing $\quad Arm_{CE} := S = 9.719\,ft$

Strength I

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

Factored vertical force

$$F_{VFtStrI} := 1.25 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \ldots$$
$$+ 1.5DW_{Sup} + 1.75R_{LLFooting} + 1.35 \cdot EV_{Ft}$$

$$F_{VFtStrI} = 3.687 \times 10^{3} \cdot kip$$

Factored shear force parallel to the transverse axis of the bridge $\quad V_{TFtStrI} := 0$

Factored shear force parallel to the longitudinal axis of the bridge $\quad V_{LFtStrI} := 1.75 \cdot BRK_{5L} = 56.875 \cdot kip$

Factored moment about the longitudinal axis of the footing

$$M_{XFtStrI} := 1.75 \cdot BRK_{5L} \cdot \left(Arm_{wall} + t_{footing}\right)$$

$$M_{XFtStrI} = 1.393 \times 10^{3} \cdot kip \cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtStrI} := 1.75 \cdot \left[\left(R_{GFt\_5L} - R_{AFt\_5L}\right) \cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right) \cdot Arm_{BF} + \left(R_{EFt\_5L} - R_{CFt\_5L}\right) \cdot Arm_{CE} \right]$$

$$M_{YFtStrI} = 894.546 \cdot kip \cdot ft$$

Strength III

Strength III = 1.25DC + 1.5DW + 1.5EH + 1.35EV + 1.0WS + 0.5TU

Factored vertical force

$$F_{VFtStrIII} := 1.25 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \dots + 1.5DW_{Sup} + 1.35 \cdot EV_{Ft}$$

$$F_{VFtStrIII} = 2.949 \times 10^{3} \cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrIII} := N_{beams} \cdot WS_{TStrIII} + P_{TPierCap.StrIII} + W_{TCol.StrIII} \cdot h_{col} + P_{TWall.StrIII} = 24.087 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrIII} := N_{beams} \cdot WS_{LStrIII} + W_{LPierCap.StrIII} \cdot l_{cap} + n_{col} \cdot W_{LCol.StrIII} \cdot h_{col} + P_{LWall.StrIII}$$

$$V_{LFtStrIII} = 34.941 \cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtStrIII} := N_{beams} \cdot WS_{LStrIII} \cdot \left(Arm_{wall} + t_{footing}\right) \dots + W_{LPierCap.StrIII} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \dots + n_{col} \cdot W_{LCol.StrIII} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.StrIII} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{XFtStrIII} = 619.589 \cdot kip \cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtStrIII} := N_{beams} \cdot WS_{TStrIII} \cdot \left(Arm_{wall} + t_{footing}\right) \dots + P_{TPierCap.StrIII} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \dots + W_{TCol.StrIII} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.StrIII} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{YFtStrIII} = 570.47 \cdot kip \cdot ft$$

Strength V

Strength V = 1.25DC + 1.5DW + 1.35LL + 1.35BR + 1.0WS + 1.0WL + 1.5EH + 1.35EV + 1.35 LS + 0.5TU

Factored vertical force

$$F_{VFtStrV} := 1.25 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \dots + 1.5DW_{Sup} + 1.35 \cdot R_{LLFooting} + 1.35 \cdot EV_{Ft}$$

$$F_{VFtStrV} = 3.518 \times 10^{3} \cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrV} := N_{beams} \cdot \left(WS_{TStrV} + WL_{TBearing}\right) + P_{TPierCap.StrV} \cdots + W_{TCol.StrV} \cdot h_{col} + P_{TWall.StrV}$$

$$V_{TFtStrV} = 26.44 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrV} := 1.35 \cdot BRK_{5L} + N_{beams} \cdot \left(WS_{LStrV} + WL_{LBearing}\right) \cdots + W_{LPierCap.StrV} \cdot l_{cap} + n_{col} \cdot W_{LCol.StrV} \cdot h_{col} + P_{LWall.StrV}$$

$$V_{LFtStrV} = 75.724 \cdot kip$$

Factored moment about the longitudinal axis of the footing

$$\begin{aligned} M_{XFtStrV} := {} & 1.35 \cdot BRK_{5L} \cdot \left(Arm_{wall} + t_{footing}\right) \cdots \\ & + N_{beams} \cdot \left[WS_{LStrV} \cdot \left(Arm_{wall} + t_{footing}\right) + WL_{LBearing} \cdot \left(Arm_{wall} + t_{footing}\right)\right] \cdots \\ & + W_{LPierCap.StrV} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots \\ & + n_{col} \cdot W_{LCol.StrV} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.StrV} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right) \end{aligned}$$

$$M_{XFtStrV} = 1.694 \times 10^{3} \cdot kip \cdot ft$$

Factored moment about the transverse axis of the footing

$$\begin{aligned} M_{YFtStrV} := {} & 1.35 \left[\begin{aligned} & \left(R_{GFt\_5L} - R_{AFt\_5L}\right) \cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right) \cdot Arm_{BF} \cdots \\ & + \left(R_{EFt\_5L} - R_{CFt\_5L}\right) \cdot Arm_{CE} \end{aligned}\right] \cdots \\ & + N_{beams} \cdot \left[WS_{TStrV} \cdot \left(Arm_{wall} + t_{footing}\right) + WL_{TBearing} \cdot \left(Arm_{wall} + t_{footing}\right)\right] \cdots \\ & + P_{TPierCap.StrV} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots \\ & + W_{TCol.StrV} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.StrV} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right) \end{aligned}$$

$$M_{YFtStrV} = 1.324 \times 10^{3} \cdot kip \cdot ft$$

Service I

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

Factored vertical force

$$F_{VFtSerI} := DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing} \cdots + DW_{Sup} + R_{LLFooting} + EV_{Ft}$$

$$F_{VFtSerI} = 2.734 \times 10^{3} \cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtSerI} := N_{beams} \cdot \left(WS_{TSerI} + WL_{TBearing}\right) + P_{TPierCap.SerI} \cdots + W_{TCol.SerI} \cdot h_{col} + P_{TWall.SerI}$$

$$V_{TFtSerI} = 22.587 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtSerI} := BRK_{5L} + N_{beams} \cdot \left(WS_{LSerI} + WL_{LBearing}\right) \cdots$$
$$+ W_{LPierCap.SerI} \cdot l_{cap} + n_{col} \cdot W_{LCol.SerI} \cdot h_{col} + P_{LWall.SerI}$$

$$V_{LFtSerI} = 58.759 \cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtSerI} := BRK_{5L} \cdot \left(Arm_{wall} + t_{footing}\right) \cdots$$
$$+ N_{beams} \cdot \left[WS_{LSerI} \cdot \left(Arm_{wall} + t_{footing}\right) + WL_{LBearing} \cdot \left(Arm_{wall} + t_{footing}\right)\right] \cdots$$
$$+ W_{LPierCap.SerI} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots$$
$$+ n_{col} \cdot W_{LCol.SerI} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.SerI} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{XFtSerI} = 1.316 \times 10^{3} \cdot kip \cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtSerI} := \left(R_{GFt\_5L} - R_{AFt\_5L}\right) \cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right) \cdot Arm_{BF} \cdots$$
$$+ \left(R_{EFt\_5L} - R_{CFt\_5L}\right) \cdot Arm_{CE} \cdots$$
$$+ N_{beams} \cdot \left[WS_{TStrV} \cdot \left(Arm_{wall} + t_{footing}\right) + WL_{TBearing} \cdot \left(Arm_{wall} + t_{footing}\right)\right] \cdots$$
$$+ P_{TPierCap.SerI} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots$$
$$+ W_{TCol.SerI} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.SerI} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{YFtSerI} = 1.141 \times 10^{3} \cdot kip \cdot ft$$

Step 5.6 Pier Cap Design

Description

This step presents the design of the pier cap. Both positive and negative moment requirements are considered.

Page Content

Design for Positive Moment

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.

It is assumed that there will be only one layer of positive moment reinforcement.

Select a trial bar size $bar := 10$

Nominal diameter of a reinforcing steel bar $d_{bar} := Dia(bar) = 1.27 \cdot in$

Cross-section area of the bar $A_{bar} := Area(bar) = 1.27 \cdot in^2$

The maximum positive moments under Strength I and Service I limit states are selected from Step 5.5.

$$M_{uPStrI} = 1.032 \times 10^3 \cdot kip \cdot ft \qquad M_{uPSerI} = 709 \cdot kip \cdot ft$$

Effective depth $d_e := h_{cap} - Cover_{cap} = 38.5 \cdot in$

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

Width of the compression face of the section $b := t_{cap} = 2.75\ ft$

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 equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

Given $M_{uPStrI} = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$ **LRFD 5.6.3.2**

Required area of steel $A_{sRequired\_cap} := Find(A_s) = 6.33 \cdot in^2$

Required number of rebars $n_{bar\_pos} := \frac{A_{sRequired\_cap}}{A_{bar}} = 4.984$

Use 5 No. 10 bars.

Number of rebars used for positive moment $n_{bar\_pos} := 5$

Area of steel provided for positive moment $A_{sProvided\_pos} := n_{bar\_pos} \cdot A_{bar} = 6.35 \cdot in^2$

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

Moment capacity of the section with the provided steel $M_{CapacityPos} := \phi_f \cdot A_{sProvided\_pos} \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_{sProvided\_pos} \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$

$$M_{CapacityPos} = 1.035 \times 10^3 \cdot kip \cdot ft$$

Distance from the extreme compression fiber to the neutral axis

$$c := \frac{A_{sProvided\_pos} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 5.33 \cdot in$$

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 be adequate to develop a factored flexural resistance at least 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 ASTM A615 Grade 60 reinforcement

Section modulus $S_c := \frac{1}{6} \cdot b \cdot h_{cap}^2 = 9.702 \times 10^3 \cdot in^3$

Concrete modulus of rupture $f_r = 0.416 \cdot ksi$

Cracking moment $M_{cr} := \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c = 360.285 \cdot kip \cdot ft$

1.33 times the factored moment demand $1.33 \cdot M_{uPStrI} = 1.373 \times 10^3 \cdot kip \cdot ft$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33M_{uPStrI}, M_{cr}\right) = 360.285 \cdot kip \cdot ft$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

Spacing requirement 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest flexural reinforcement $d_c := Cover_{cap} = 3.5 \cdot in$

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

$$\beta_s := 1 + \frac{d_c}{0.7\left(h_{cap} - d_c\right)} = 1.13$$

The position of the cross-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 $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvided\_pos} \cdot (d_e - x)$

Position of the neutral axis $x_{na} := Find(x) = 9.456 \cdot in$

Tensile force in the reinforcing steel due to service limit state moment $T_s := \dfrac{M_{uPSerI}}{d_e - \frac{x_{na}}{3}} = 240.7 \cdot kip$

Stress in the reinforcing steel due to service limit state moment $f_{ss1} := \dfrac{T_s}{A_{sProvided\_pos}} = 37.904 \cdot ksi$

f_{ss} (not to exceed $0.6f_y$) $f_{ss} := min(f_{ss1}, 0.6f_y) = 36 \cdot ksi$

Required reinforcement bar spacing $s_{barRequred} := \dfrac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 10.209 \cdot in$

Spacing of the steel reinforcement bars $s_{bar} := \dfrac{(b - 2 \cdot Cover_{cap})}{4} = 6.5 \cdot in$

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

Shrinkage and Temperature Reinforcement Requirement

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

For bars, the area of reinforcing steel per-foot (A_s), on each face and in each direction, shall satisfy $A_S \geq \dfrac{1.3bh}{2(b + h)f_y}$ **LRFD 5.10.6**

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

The following calculation evaluates the above limits to identify the minimum area of shrinkage and temperature reinforcement needed for the pier cap.

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := min\left[\begin{bmatrix} \left(0.60 \frac{in^2}{ft}\right) \\ max\left[\begin{bmatrix} \left(0.11 \frac{in^2}{ft}\right) \\ \left[\dfrac{1.3 \cdot h_{cap} \cdot t_{cap} \cdot \frac{kip}{in \cdot ft}}{2(h_{cap} + t_{cap}) \cdot f_y}\right] \end{bmatrix}\right] \end{bmatrix}\right] = 0.2 \cdot \frac{in^2}{ft}$$

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

Design for Negative Moment

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.

It is assumed that there will be only one layer of negative moment reinforcement.

Select a trial bar size $bar := 8$

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

Cross-section area of the bar $A_{bar} := Area(bar) = 0.79 \cdot in^2$

The maximum negative moments under Strength I and Service I limit states are obtained from Step 5.5.

$$M_{uNStrI} = 680.7 \cdot kip \cdot ft \qquad M_{uNSerI} = 473.2 \cdot kip \cdot ft$$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

Given $$M_{uNStrI} = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right]$$ **LRFD 5.6.3.2**

Required area of steel $A_{sRequired\_neg} := Find(A_s) = 4.083 \cdot in^2$

Required number of rebars $$n_{bar\_neg} := \frac{A_{sRequired\_neg}}{A_{bar}} = 5.169$$

Use 6 No. 8 bars.

Number of rebars used for negative moment $n_{bar\_neg} := 6$

$$A_{sProvided\_neg} := n_{bar\_neg} \cdot A_{bar} = 4.74 \cdot in^2$$

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

Moment capacity of the section with the provided steel $$M_{Capacity\_neg} := \phi_f \cdot A_{sProvided\_neg} \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_{sProvided\_neg} \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right]$$

$$M_{Capacity\_neg} = 785.161 \cdot kip \cdot ft$$

Distance from the extreme compression fiber to the neutral axis $$c := \frac{A_{sProvided\_neg} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 3.98 \cdot in$$

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 be adequate to develop a factored flexural resistance at least 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 ASTM A615 Grade 60 reinforcement

Section modulus $S_c := \frac{1}{6} \cdot b \cdot h_{cap}{}^2 = 9.702 \times 10^3 \cdot in^3$

Cracking moment $M_{cr} := \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c = 360.285 \cdot kip \cdot ft$

1.33 times the factored moment demand $1.33 \cdot M_{uNStrI} = 905.331 \cdot kip \cdot ft$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33 M_{uNStrI}, M_{cr}\right) = 360.285 \cdot kip \cdot ft$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

Spacing requirement 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest flexural reinforcement $d_c := Cover_{cap} = 3.5 \cdot in$

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

$$\beta_s := 1 + \frac{d_c}{0.7\left(h_{cap} - d_c\right)} = 1.13$$

The position of the cross-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 $x := 6in$

Given

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

Position of the neutral axis $x_{na} := Find(x) = 8.327 \cdot in$

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

$$T_s := \frac{M_{uNSerI}}{d_e - \frac{x_{na}}{3}} = 159 \cdot kip$$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided\_neg}} = 33.534 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min(f_{ss1}, 0.6f_y) = 33.534 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequred} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 11.475 \cdot in$$

Spacing of the steel reinforcement bars

$$s_{bar} := \frac{(b - 2 \cdot Cover_{cap})}{4} = 6.5 \cdot in$$

Check if the spacing provided < the required spacing

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

Shrinkage and Temperature Reinforcement Requirement

LRFD 5.10.6

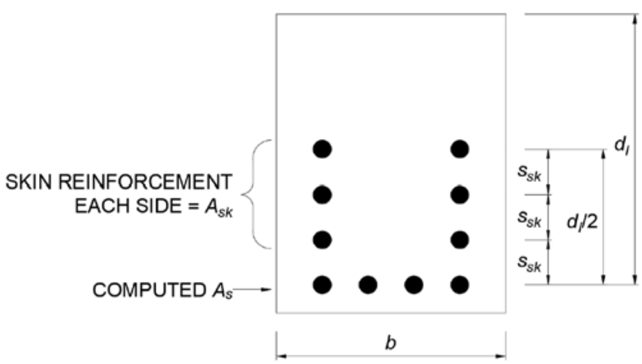
Check if the provided area of steel > the required area of shrinkage and temperature steel

$$Check := if(A_{sProvided\_neg} > A_{shrink.temp} \cdot t_{cap}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$$

Design for Skin Reinforcement

LRFD 5.6.7

Concrete flexural members with depths exceeding 3 ft have a tendency to develop excessively wide cracks in the upper parts of their tension zones. To reduce the width of these cracks, it is necessary to provide additional longitudinal reinforcing steel in the zone of flexural tension near the vertical side faces of their web. This additional steel, which is referred to as the longitudinal skin reinforcement, must be uniformly distributed along both side faces for a distance equal to d/2 closer to the flexural reinforcing steel, as shown below.



Distance from the extreme compression fiber to the centroid of the extreme tension steel

$$d_l := h_{cap} - Cover_{cap} = 38.5 \cdot in$$

The maximum spacing of skin reinforcement

$$s_{minSkin} := \min\left(\frac{d_l}{6}, 12in\right) = 6.417 \cdot in$$

Required area of skin reinforcement on each side face of the pier cap

$$A_{sk1} := 0.012 \cdot (d_l - 30in) \cdot \frac{in}{ft} = 0.102 \cdot \frac{in^2}{ft}$$

LRFD Eq. 5.6.7-3

One fourth of the required flexural tensile reinforcement

$$A_{sk2} := \frac{1}{4} \cdot \frac{A_{sRequired\_cap}}{0.5d_l} = 0.987 \cdot \frac{in^2}{ft}$$

The required area of skin reinforcement on each side face of the pier cap not to exceed one fourth of the flexural tensile reinforcement

$$A_{sk\_required} := \min\left(A_{sk1}, A_{sk2}\right) = 0.102 \cdot \frac{in^2}{ft}$$

Select a trial bar size for each side face

$$bar := 4$$

Select a spacing for reinforcing steel bars

$$S_{bar} := 6in$$

Cross-section area of a reinforcing bar

$$A_{bar} := Area(bar) = 0.2 \cdot in^2$$

Area of skin reinforcement provided on each side face of the pier cap

$$A_{sk\_provided} := A_{bar} \cdot \frac{\frac{12in}{S_{bar}}}{ft} = 0.4 \cdot \frac{in^2}{ft}$$

Check if $A_{sk\_provided} > A_{sk\_required}$

$$Check := if\left(A_{sk\_provided} > A_{sk\_required}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

The above calculations checked skin reinforcement requirements in the positive moment region. For the negative moment region, providing No. 4 bars at 6 in. spacing meets the requirement. Therefore, providing 5 No. 4 bars evenly distributed on each side face of the pier cap satisfies the skin reinforcement requirement.

Actual spacing of the skin reinforcement

$$s_{skin} := \frac{h_{cap} - 2 \cdot Cover_{cap}}{6} = 5.833 \cdot in$$

Check if the actual spacing of the skin reinforcement < the selected spacing

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

Design for Shear

A simplified design procedure can be used since the section is not subjected to axial tension and contains at least the minimum amount of transverse reinforcement. **LRFD 5.7.3.4.1**

Considering symmetry, the left half of the pier cap is designed. The structural analysis results show that the maximum factored shear force is at the pier cap section located between Girder B and Column 2. The maximum shear is located at the face of Column 2.

Effective width of the section

$$b_v := b = 33 \cdot in$$

Depth of equivalent rectangular stress block

$$a := \frac{A_{sProvided\_neg} \cdot f_y}{0.85 \cdot f_c \cdot b} = 3.38 \cdot in$$

Effective shear depth

$$d_v := \max\left(d_e - \frac{a}{2}, 0.9 \cdot d_e, 0.72 \cdot h_{cap}\right) = 36.81 \cdot in$$

LRFD 5.7.2.8

The critical section for shear should be taken as d_v from the internal face of Column 2. Note that if a bridge bearing is located within d_v from the face of the column, the critical section should be at the face of the column. In this example, the bearings are not located within d_v from the face of the columns. **LRFD 5.7.3.2**

Maximum factored shear at the critical section (Demand) (from structural analysis)

$$V_{uCapStrI} := 467.8kip$$

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

$$\beta := 2$$

Angle of inclination of diagonal compressive stresses

$$\theta := 45$$

Nominal shear resistance of concrete $V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_v = 133 \cdot kip$ **LRFD Eq. 5.7.3.3-3**

Resistance factor for shear (for normal weight concrete) $\phi_v := 0.9$ **LRFD 5.5.4.2**

Shear stress on the concrete $v_u := \dfrac{V_{uCapStrI}}{\phi_v \cdot b_v \cdot d_v} = 0.428 \cdot ksi$ **LRFD Eq. 5.7.2.8-1**

$$Check := if\left(v_u < 0.125 \cdot f_c, \text{"Max. spacing = 24 in."}, \text{"Max. spacing = 12 in."}\right) = \text{"Max. spacing = 12 in."}$$

The maximum spacing of the transverse reinforcement shall not exceed 12 in. **LRFD 5.7.2.6**

Select trial stirrup size and number of legs $bar := 5$ $leg := 6$

Select stirrup spacing $s := 6in$

Cross-section area of one leg of a stirrup $A_{bar} := Area(bar) = 0.31 \cdot in^2$

Total stirrup area $A_v := leg \cdot A_{bar} = 1.86 \cdot in^2$

Check minimum transverse (shear) reinforcement requirement $0.0316 \cdot \beta \cdot \dfrac{\sqrt{f_c \cdot ksi} \cdot b \cdot s}{f_y} = 0.361 \cdot in^2$ **LRFD Eq. 5.7.2.5-1**

$$Check := if\left(A_v > 0.0316 \cdot \beta \cdot \frac{\sqrt{f_c \cdot ksi} \cdot b \cdot s}{f_y}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shear resistance provided by stirrups $V_s := \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s} = 422.694 \cdot kip$ **LRFD Eq. 5.7.3.3-4**

The nominal shear resistance, V_n, at the critical section is calculated as follows:

$$V_{n1} := V_c + V_s = 555.666 \cdot kip$$

$$V_{n2} := 0.25 f_c \cdot b \cdot d_v = 911.051 \cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := \min\left(V_{n1}, V_{n2}\right) = 555.666 \cdot kip$$

Factored shear resistance (Capacity) $V_r := \phi_v \cdot V_n = 500.099 \cdot kip$

Check if the shear capacity i> the shear demand $Check := if\left(V_r > V_{uCapStrI}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Use 6 legs of No. 5 stirrups at 6 in. spacing for the pier cap segment located between Girder B and Column 2.

For the pier cap segment located between Column 2 and Girder C:

Maximum factored shear force at the critical section (Demand) (from structural analysis) $V_{u2C} := 405.1kip$

Shear stress on the concrete $v_u := \dfrac{V_{u2C}}{\phi_v \cdot b_v \cdot d_v} = 0.371 \cdot ksi$ **LRFD Eq. 5.7.2.8-1**

$$Check := if\left(v_u < 0.125 \cdot f_c, \text{"Max. spacing = 24 in."}, \text{"Max. spacing = 12 in."}\right) = \text{"Max. spacing = 24 in."}$$

The maximum spacing of the transverse reinforcement shall not exceed 24 in. **LRFD 5.7.2.6**

Since the maximum factored shear force is close to the section located between Girder B and Column 2, use the same amount of transverse reinforcement.

Use 6 legs of No. 5 stirrups at 6 in. spacing for the pier cap segment located between Column 2 and Girder C.

For the pier cap segment located between Girder C and Girder E:

Maximum factored shear force at the critical section (Demand) (from structural analysis) $V_{uCE} := 154.9kip$ $>$ $\phi_v \cdot V_c = 119.675 \cdot kip$

Shear stress on the concrete $v_u := \dfrac{V_{uCE}}{\phi_v \cdot b_v \cdot d_v} = 0.142 \cdot ksi$ **LRFD Eq. 5.7.2.8-1**

$Check := if\left(v_u < 0.125 \cdot f_c, \text{"Max. spacing = 24 in."}, \text{"Max. spacing = 12 in."}\right) = \text{"Max. spacing = 24 in."}$

The maximum spacing of the transverse reinforcement shall not exceed 24 in. **LRFD 5.7.2.6**

Select trial stirrup size and number of legs $bar := 5$ $leg := 4$

Select a spacing for stirrups $s := 12in$

Total stirrup area $A_v := leg \cdot Area(bar) = 1.24 \cdot in^2$

Check the minimum transverse (shear) reinforcement requirement $0.0316 \cdot \beta \cdot \dfrac{\sqrt{f_c \cdot ksi} \cdot b \cdot s}{f_y} = 0.722 \cdot in^2$ **LRFD Eq. 5.7.2.5-1**

$$Check := if\left(A_v > 0.0316 \cdot \beta \cdot \frac{\sqrt{f_c \cdot ksi} \cdot b \cdot s}{f_y}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shear resistance provided by stirrups $V_s := \dfrac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s} = 140.898 \cdot kip$ **LRFD Eq. 5.7.3.3-4**

The nominal shear resistance, V_n, at the critical section is calculated as follows:

$$V_{n1} := V_c + V_s = 273.87 \cdot kip$$

$$V_{n2} := 0.25 f_c \cdot b \cdot d_v = 911.051 \cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := \min\left(V_{n1}, V_{n2}\right) = 273.87 \cdot kip$$

Factored shear resistance (Capacity) $V_r := \phi_v \cdot V_n = 246.483 \cdot kip$

Check if the shear capacity > the shear demand $Check := if\left(V_r > V_{uCE}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Use 4 legs of No. 5 stirrups at 12 in. spacing for the pier cap segment located between Girder C and Girder E.

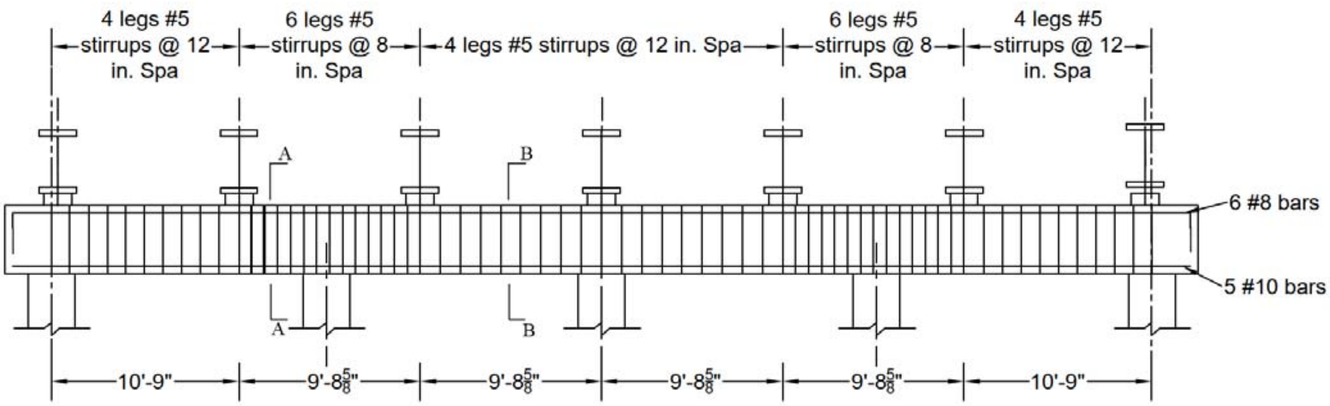
For the pier cap segment located between Column 1 and Girder B:

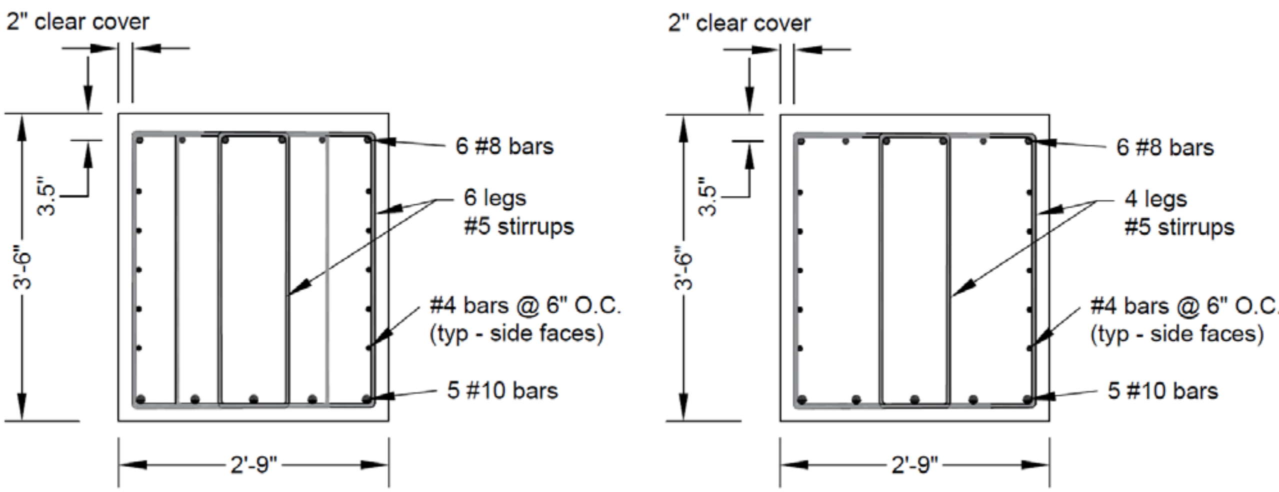
Maximum factored shear force at the critical section (Demand) (from structural analysis) $V_{u1B} := 111.6kip$ $<$ $\phi_v \cdot V_c = 119.675 \cdot kip$

Therefore, use the minimum transverse reinforcement requirement. Considering constructability, use the same shear reinforcement as in the section between Girder C and E.

Use 4 legs of No. 5 stirrups at 12 in. spacing for the pier cap segment located between Column 1 and Girder B.

The following figures present the pier cap design details:





Section A-A

Section B-B

Note: Certain details are not shown in this drawing for clarity of the main reinforcement. Refer to the MDOT Bridge Design Guides for additional details.

Step 5.7 Pier Column Design

Description

This step presents the design of the pier columns.

Page **Content**

Design Forces and Moments

The following forces and moments are obtained from Step 5.5:

Strength I Limit State

Axial load at the critical section $P_{u\_StrI} = 862.6 \cdot kip$

Moment about the longitudinal axis of the bridge $M_{ut\_StrI} = 159.2 \cdot kip \cdot ft$

Moment about the transverse axis of the bridge $M_{ul\_StrI} = 147.34 \cdot kip \cdot ft$

Shear parallel to the longitudinal axis of the bridge $V_{ul\_StrI} = 10.1 \cdot kip$

Shear parallel to the transverse axis of the bridge $V_{ut\_StrI} = 20.6 \cdot kip$

Strength V Limit State

Axial load at the critical section $P_{u\_StrV} = 780.8 \cdot kip$

Moment about the longitudinal axis of the bridge $M_{ut\_StrV} = 165.8 \cdot kip \cdot ft$

Moment about the transverse axis of the bridge $M_{ul\_StrV} = 176 \cdot kip \cdot ft$

Shear parallel to the longitudinal axis of the bridge $V_{ul\_StrV} = 11.7 \cdot kip$

Shear parallel to the transverse axis of the bridge $V_{ut\_StrV} = 23.3 \cdot kip$

Preliminary Design of the Column

Column diameter $d_{col} = 2.5\ ft$

Column gross cross-section area $A_{g\_col} := \pi \cdot \frac{d_{col}^2}{4} = 4.909\ ft^2$

The minimum number of longitudinal reinforcing bars in the body of a column shall be six; they shall be placed in a circular arrangement. The minimum size of a bar shall be No. 5. **LRFD 5.6.4.2**

Select a trial bar size for vertical reinforcement $bar := 9$

Nominal diameter of a vertical reinforcing steel bar $d_{bar} := Dia(bar) = 1.128 \cdot in$

Cross-section area of a vertical reinforcing steel bar $A_{bar} := Area(bar) = 1 \cdot in^2$

Select the number of vertical reinforcing bars $N_{bar} := 8$

Total area of vertical reinforcement $A_{s\_col} := N_{bar} \cdot A_{bar} = 8 \cdot in^2$

Select a spacing for transverse ties $s_{tie} := 12in$

Limits for Reinforcement in Compression Members

LRFD 5.6.4.2

$$\frac{A_{s\_col}}{A_{g\_col}} = 0.011$$

Check the maximum steel reinforcement limit

$$Check := if\left(\frac{A_{s\_col}}{A_{g\_col}} \le 0.08, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

LRFD Eq. 5.6.4.2-1

Check the minimum steel reinforcement limit

$$Check := if\left(\frac{A_{s\_col}}{A_{g\_col}} \ge 0.135 \cdot \frac{f_c}{f_y}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

LRFD Eq. 5.6.4.2-3

Slenderness Effects

LRFD 5.6.4.3

The unbraced column length used for calculating the slenderness ratio about each direction is the full column height, which is the height from the top of the base wall to the bottom of the pier cap. It is assumed that the superstructure has no effect on restraining the pier from column buckling. The pier is considered a free-standing cantilever in the longitudinal direction of the bridge. Therefore, the effective length factor in the longitudinal direction of the bridge is 2.1. The effective length factor in the transverse direction of the bridge is taken as 1.2 to account for the high rigidity of the pier cap.

Radius of gyration

$$r_{col} := \frac{1}{4} \cdot d_{col} = 0.625\ ft$$

Effective length factor in the longitudinal direction of the bridge

$$K_l := 2.1$$

Effective length factor in the transverse direction of the bridge

$$K_t := 1.2$$

Slenderness ratios in the longitudinal and transverse directions of the bridge

$$\frac{K_l \cdot h_{col}}{r_{col}} = 40.32 \qquad \frac{K_t \cdot h_{col}}{r_{col}} = 23.04$$

For members that are not braced against sidesway, the effect of slenderness may be neglected where the slenderness ratio is less than 22. If the slenderness ratio is less than 100, the approximate procedure may be used for the design of non-prestressed compression members.

LRFD 5.6.4.3

Moment Magnification Method

The moment magnification method is used to approximate the column moments. To calculate the amplification factor, the column stiffness (EI) needs to be defined. In doing so, the ratio of the maximum factored permanent load moment to the maximum factored total load moment must be identified.

Strength I Limit State

Moment about the Transverse Direction of the Bridge

The following factored moment under dead loads is calculated using a structural analysis software.

Maximum factored permanent load moment at the critical section

$$M_{l\_permanent\_StrI} := 0 kip \cdot ft$$

Maximum factored total load moment at the critical section

$$M_{ul\_StrI} = 147.34 \cdot kip \cdot ft$$

Ratio of the maximum factored permanent load moments to the maximum factored total load moment

$$\beta_{dl} := \frac{M_{l\_permanent\_StrI}}{M_{ul\_StrI}} = 0$$

Moment of inertia of the gross concrete section about the centroidal axis

$$I_g := \frac{1}{4}\pi\left(\frac{d_{col}}{2}\right)^4 = 3.976 \times 10^4 \cdot in^4$$

As a simplification, steel reinforcement in the column is not considered for EI calculation. Therefore, the column stiffness is:

$$EI := \frac{\frac{E_c \cdot I_g}{2.5}}{\left(1 + \beta_{dl}\right)} = 5.766 \times 10^7 \cdot kip \cdot in^2$$ **LRFD Eq. 5.6.4.3-2**

Resistance factor for compression $\phi_{axial} := 0.75$

Stiffness reduction factor for concrete members $\phi_k := 0.75$ **LRFD 4.5.3.2.2b**

Euler buckling load

$$P_{el} := \frac{\pi^2 \cdot EI}{\left(K_l \cdot h_{col}\right)^2} = 6.223 \times 10^3 \cdot kip$$ **LRFD Eq. 4.5.3.2.2b-5**

For a conservative design, assume that the sidesway of the pier in the transverse direction of the bridge is significant.

Moment magnification factor

$$\delta_{sl} := \frac{1}{1 - \left(\frac{P_{u\_StrI}}{\phi_k \cdot P_{el}}\right)} = 1.227$$ **LRFD Eq. 4.5.3.2.2b-4**

As a simplification, use the same magnification factor for column moments due to gravity and lateral loads.

Magnified moment about the transverse direction of the bridge

$$M_{cl\_StrI} := \delta_{sl} \cdot M_{ul\_StrI} = 180.744 \cdot kip \cdot ft$$

Moment about the Longitudinal Direction of the Bridge

The following factored moment under dead loads is calculated using a structural analysis software.

Maximum factored permanent load moment at the critical section $M_{t\_permanent\_StrI} := 81.4 kip \cdot ft$

Maximum factored total load moment at the critical section $M_{ut\_StrI} = 159.2 \cdot kip \cdot ft$

Ratio of the maximum factored permanent load moments to the maximum factored total load moment

$$\beta_{dt} := \frac{M_{t\_permanent\_StrI}}{M_{ut\_StrI}} = 0.511$$

As a simplification, steel reinforcement in the column is not considered for EI calculation. Therefore, the column stiffness is:

$$EI := \frac{\frac{E_c \cdot I_g}{2.5}}{\left(1 + \beta_{dt}\right)} = 3.815 \times 10^7 \cdot kip \cdot in^2$$ **LRFD Eq. 5.6.4.3-2**

Euler buckling load

$$P_{et} := \frac{\pi^2 \cdot EI}{\left(K_t \cdot h_{col}\right)^2} = 1.261 \times 10^4 \cdot kip$$ **LRFD Eq. 4.5.3.2.2b-5**

For a conservative design, assume that the sidesway of the pier in the longitudinal direction of the bridge is significant.

Moment magnification factor

$$\delta_{st} := \frac{1}{1 - \left(\frac{P_{u\_StrI}}{\phi_k \cdot P_{et}}\right)} = 1.1$$

LRFD Eq. 4.5.3.2.2b-4

As a simplification, use the same magnification factor for column moments due to gravity and lateral loads.

Magnified moment about the longitudinal direction of the bridge

$$M_{ct\_StrI} := \delta_{st} \cdot M_{ut\_StrI} = 175.177 \cdot kip \cdot ft$$

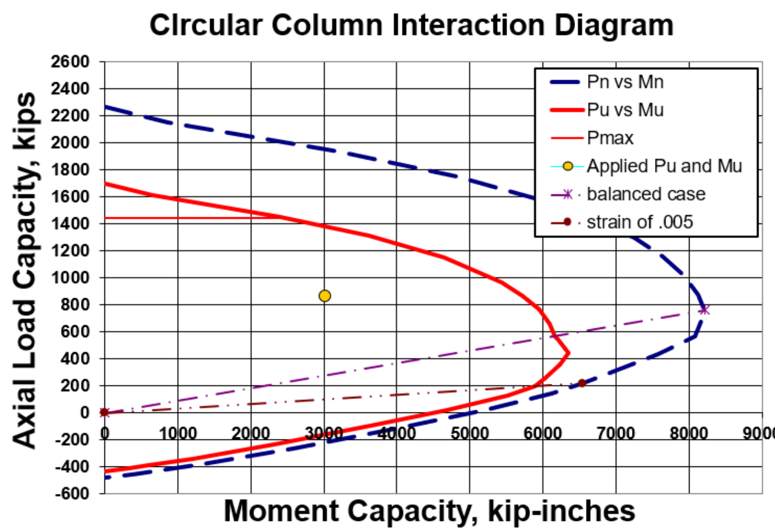
The combined moment

$$M_{u\_StrI} := \sqrt{M_{cl\_StrI}^2 + M_{ct\_StrI}^2} = 3.02 \times 10^3 \cdot kip \cdot in$$

Factored axial load

$$P_{u\_StrI} = 862.6 \cdot kip$$

The interaction diagram of the column is developed using a software program. As shown in the diagram, the applied factored loads (the demand, marked using an orange dot) are lower than the capacity.



Strength V Limit State

<u>Moment about the Transverse Direction of the Bridge</u>

The following factored moment under dead loads is calculated using a structural analysis software.

Maximum factored permanent load moment at the critical section

$$M_{l\_permanent\_StrV} := 0 kip \cdot ft$$

Maximum factored total load moment at the critical section

$$M_{ul\_StrV} = 176 \cdot kip \cdot ft$$

Ratio of the maximum factored permanent load moments to the maximum factored total load moment

$$\beta_{dl} := \frac{M_{l\_permanent\_StrV}}{M_{ul\_StrV}} = 0$$

As a simplification, steel reinforcement in the column is not considered for the EI calculation. Therefore, the column stiffness is:

$$EI := \frac{\frac{E_c \cdot I_g}{2.5}}{\left(1 + \beta_{dl}\right)} = 5.766 \times 10^7 \cdot kip \cdot in^2$$

LRFD Eq. 5.6.4.3-2

Euler buckling load

$$P_{el} := \frac{\pi^2 \cdot EI}{\left(K_l \cdot h_{col}\right)^2} = 6.223 \times 10^3 \cdot kip$$

LRFD Eq. 4.5.3.2.2b-5

For a conservative design, assume that the sidesway of the pier in the transverse direction of the bridge is significant.

Moment magnification factor $\delta_{sl} := \dfrac{1}{1 - \left(\dfrac{P_{u\_StrV}}{\phi_k \cdot P_{el}}\right)} = 1.201$ **LRFD Eq. 4.5.3.2.2b-4**

As a simplification, use the same magnification factor for column moment due to gravity and lateral loads.

Magnified moment about the transverse direction of the bridge $M_{cl\_StrV} := \delta_{sl} \cdot M_{ul\_StrV} = 211.357 \cdot kip \cdot ft$

Moment about the Longitudinal Direction of the Bridge

The following factored moment under dead loads is calculated using a structural analysis software.

Maximum factored permanent load moment at the critical section $M_{t\_permanent\_StrV} := 81.4 kip \cdot ft$

Maximum factored total load moment at the critical section $M_{ut\_StrV} = 165.8 \cdot kip \cdot ft$

Ratio of the maximum factored permanent load moments to the maximum factored total load moment $\beta_{dt} := \dfrac{M_{t\_permanent\_StrI}}{M_{ut\_StrI}} = 0.511$

As a simplification, steel reinforcement in the column is not considered for the EI calculation. Therefore, the column stiffness is:

$$EI := \frac{\dfrac{E_c \cdot I_g}{2.5}}{\left(1 + \beta_{dt}\right)} = 3.815 \times 10^7 \cdot kip \cdot in^2$$ **LRFD Eq. 5.6.4.3-2**

Euler buckling load $P_{et} := \dfrac{\pi^2 \cdot EI}{\left(K_t \cdot h_{col}\right)^2} = 1.261 \times 10^4 \cdot kip$ **LRFD Eq. 4.5.3.2.2b-5**

For a conservative design, assume that the sidesway of the pier, in the longitudinal direction of the bridge, is significant.

Moment magnification factor $\delta_{st} := \dfrac{1}{1 - \left(\dfrac{P_{u\_StrV}}{\phi_k \cdot P_{et}}\right)} = 1.09$ **LRFD Eq. 4.5.3.2.2b-4**

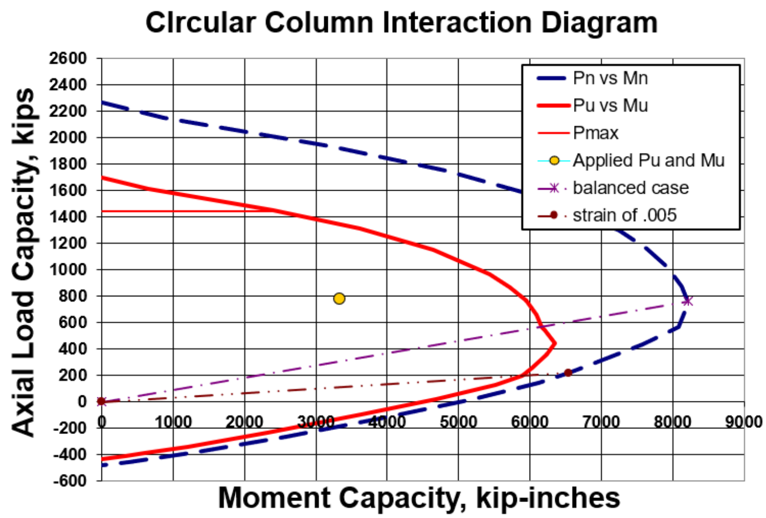
As a simplification, use the same magnification factor for column moments due to gravity and lateral loads.

Magnified moment about the longitudinal direction of the bridge $M_{ct\_StrV} := \delta_{st} \cdot M_{ut\_StrV} = 180.719 \cdot kip \cdot ft$

Combined moment $M_{u\_StrV} := \sqrt{M_{cl\_StrV}{}^2 + M_{ct\_StrV}{}^2} = 3.337 \times 10^3 \cdot kip \cdot in$

Factored axial load $P_{u\_StrV} = 780.8 \cdot kip$

The interaction diagram of the column is developed using a software program. As shown in the diagram, the applied factored loads (the demand, marked using an orange dot) are lower than the capacity.



Design for Shear

After examination of the load effects, Strength V is identified as the controlling limit state for the shear force acting on the columns.

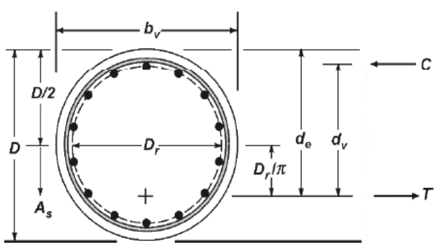
Shear parallel to the transverse axis of the bridge $V_{ut\_StrV} = 23.3 \cdot kip$

Shear parallel to the longitudinal axis of the bridge $V_{ul\_StrV} = 11.7 \cdot kip$

Controlling factored shear $V_{uCol} := \sqrt{V_{ut\_StrV}^2 + V_{ul\_StrV}^2} = 26.073 \cdot kip$

The simplified design procedure can be used since (1) there is no axial tension and (2) the minimum amount of ties is provided in the column. **LRFD 5.7.3.4.1**

For circular members, the terms b_v, d_v, d_e are defined as shown below.



LRFD Figure C5.7.2.8-2

Diameter of the column $D := d_{col} = 2.5\ ft$

Diameter of the circle passing through the centers of the longitudinal reinforcement $D_r := D - 2 \cdot Cover_{col} = 22 \cdot in$

Effective width of the section $b_v := D = 30 \cdot in$

Effective depth from extreme compression fiber to the centroid of the tensile force in the tensile reinforcement $d_e := \frac{D}{2} + \frac{D_r}{\pi} = 22.003 \cdot in$ **LRFD Eq. C5.7.2.8-2**

Effective shear depth $d_v := \max\left(d_e - Cover_{col}, 0.9 d_e, 0.72 D\right) = 1.8\ ft$

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

Nominal shear resistance of concrete $V_c := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b_v \cdot d_v = 70.9 \cdot kip$ **LRFD Eq. 5.7.3.3-3**

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

$$V_{uCol} = 26.073 \cdot kip \ < \ 0.5\phi_v \cdot V_c = 31.92 \cdot kip$$

Since the shear demand (V_{uCol}) is less than $0.5\phi_v V_c$, the transverse reinforcement is not required. **LRFD 5.7.2.3**

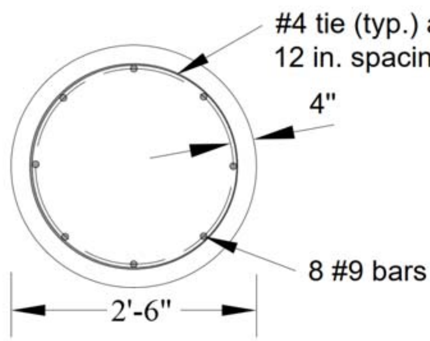
Although the transverse reinforcement is not required for these columns, transverse confinement steel in the form of hoops, ties, or spirals is required for compression members.

Select No. 4 bars as ties. **BDM 7.04.01 G**

Note: No. 4 is the minimum bar size MDOT uses to avoid damages during shipping and handling.

The spacing of ties along the longitudinal axis of the columns with single bars shall not exceed the lesser of the least dimension of the member or 12.0 in. Therefore, use 12 in. spacing. **LRFD 5.10.4.3**

Ties shall be located vertically; they must not be placed more than half a tie spacing above the base wall and not more than half a tie spacing below the pier cap. **LRFD 5.10.4.3**



Note: Tie lap is not shown in the figure for clarity.

Step 5.8 Base Wall Design

Description

This step presents the structural design of the pier base wall.

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From Step 5.5, the controlling moment and shear applied on the base wall are

$$V_{Wall} = 75.724 \cdot kip \qquad M_{Wall} = 1.467 \times 10^{3} \cdot kip \cdot ft$$

On a per-foot basis $\quad V_{uWall} := \dfrac{V_{Wall}}{l_{wall}} = 1.221 \cdot \dfrac{kip}{ft} \qquad M_{uWall} := \dfrac{M_{Wall}}{l_{wall}} = 23.656 \cdot \dfrac{kip \cdot ft}{ft}$

For Service I limit state $\quad M_{uWallSerI} := \dfrac{M_{WallSerI}}{l_{wall}} = 19.255 \cdot \dfrac{kip \cdot ft}{ft}$

Design for Flexure

Flexural 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.

Select a trial bar size $\quad bar := 6$

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

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

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. **BDG 6.20.03 and 6.20.03A**

Wall thickness $\quad t_{wall} = 36 \cdot in$

Select a spacing for reinforcing steel bars $\quad s_{bar} := 12 \cdot in$

Reinforcing steel area provided in the wall $\quad A_{sProvided} := \dfrac{A_{bar} \cdot 12in}{s_{bar}} = 0.44 \cdot in^{2}$

Effective depth $\quad d_{e} := t_{wall} - Cover_{wall} = 32.25 \cdot in$

Resistance factor for flexure $\quad \phi_{f} := 0.9$ **LRFD 5.5.4.2**

Select a 1-ft wide strip for the design.

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

Stress block factor $\quad \beta_{1} = 0.85$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

Given $M_{uWall} \cdot ft = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$ **LRFD 5.6.3.2**

Required area of steel $A_{sRequired} := Find(A_s) = 0.164 \cdot in^2$

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

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

$$M_{CapacityWall} = 63.001 \cdot \frac{kip \cdot ft}{ft}$$

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

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must be adequate to develop a factored flexural resistance at least 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 ASTM A615 Grade 60 reinforcement

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

Concrete modulus of rupture $f_r = 0.416 \cdot ksi$

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 $1.33 \cdot M_{uWall} = 31.462 \cdot \frac{kip \cdot ft}{ft}$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := min(1.33 \cdot M_{uWall}, M_{cr}) = 31.462 \cdot \frac{kip \cdot ft}{ft}$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

Spacing requirement 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 the Class 1 exposure condition

$$\gamma_e := 1.00$$

Distance from extreme tension fiber to center of the closest bar

$$d_c := Cover_{wall} = 3.75 \cdot in$$

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

$$\beta_s := 1 + \frac{d_c}{0.7\left(t_{wall} - d_c\right)} = 1.166$$

The position of the cross-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

$$x_{na} := Find(x) = 4.066 \cdot in$$

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

$$T_s := \frac{M_{uWallSerI}}{d_e - \frac{x_{na}}{3}} \cdot ft = 7.5 \cdot kip$$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided}} = 16.998 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 16.998 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequred} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 27.816 \cdot in$$

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

LRFD 5.10.6

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

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := \min\left[\begin{bmatrix} \left(0.60\,\frac{in^2}{ft}\right) \\ \max\left[\begin{bmatrix} \left(0.11\,\frac{in^2}{ft}\right) \\ \left[\frac{1.3\cdot h_{wall}\cdot t_{wall}\cdot\frac{kip}{in\cdot ft}}{2\left(h_{wall}+t_{wall}\right)\cdot f_y}\right]\end{bmatrix}\right]\end{bmatrix}\right]\cdot ft = 0.26\cdot in^2$$

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

The factored shear force applied in the transverse direction of the wall on a per-foot basis

$$V_{uWall} = 1.221\cdot\frac{kip}{ft}$$

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

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

Effective shear depth $\quad d_v := \max\left(d_e - \dfrac{a}{2}, 0.9\cdot d_e, 0.72\cdot t_{wall}\right) = 31.819\cdot in \quad$ **LRFD 5.7.2.8**

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

$$V_{c1} := 0.0316\cdot\beta\cdot\sqrt{f_c\cdot ksi}\cdot b\cdot d_v = 41.8\cdot kip$$ **LRFD Eq. 5.7.3.3-3**

$$V_{c2} := 0.25 f_c\cdot b\cdot d_v = 286.368\cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := \min\left(V_{c1}, V_{c2}\right) = 41.797\cdot kip$$

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

Factored shear resistance (Capacity) $\quad V_r := \phi_v\cdot V_n = 37.617\cdot kip$

Check if the shear capacity > the shear demand

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

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 $l_{db} := 2.4 \cdot d_{bar} \cdot \frac{f_y}{\sqrt{f_c \cdot ksi}} = 5.196\ ft$ **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 $c_b := Cover_{wall} = 3.75 \cdot in$

Reinforcement confinement factor $\lambda_{rc} := \frac{d_{bar}}{c_b} = 0.2$

Excess reinforcement factor $\lambda_{er} := \frac{A_{sRequired}}{A_{sProvided}} = 0.372$

Factor for normal weight concrete $\lambda := 1$

LRFD Eq. 5.10.8.2.1a-1

Required development length $l_{d.required} := l_{db} \cdot \frac{\left(\lambda_{rl} \cdot \lambda_{cf} \cdot \lambda_{rc} \cdot \lambda_{er}\right)}{\lambda} = 6.965 \cdot in$

Available length length for rebar development $l_{d.available} := t_{footing} - Cover_{ft} = 32 \cdot in$

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

Since the footing thickness is 3 ft, adequate space is available for straight bars.

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 base wall.

The horizontal reinforcement at each face of the wall should satisfy the shrinkage and temperature reinforcement requirements. **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**

Note: MDOT uses No. 6 bars at a maximum spacing of 18 in. **BDG 5.22.01**

Select a trial bar size $bar := 6$

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

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

Select a spacing for reinforcing steel bars $s_{barST} := 18 \cdot in$

Reinforcing steel area provided in the wall $A_{sProvidedST} := \frac{A_{bar} \cdot 12in}{s_{barST}} = 0.293 \cdot in^2$

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

Required minimum shrinkage and temperature steel area

$$A_{shrink.temp} = 0.26 \cdot in^2$$

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

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

The base wall design presented in this step resulted in the following details:

- No. 6 bars @ 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) on each face of the wall as the vertical flexural reinforcement
- No. 6 bars @ 18.0 in. spacing ($A_s = 0.293$ in.^{2}/ft) on each face of the wall as the horizontal shrinkage and temperature reinforcement.

Step 5.9 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

Step 5.10 presents the structural design of the footing.

Page Content

Bearing Resistance Check

For eccentrically loaded footings, the use of a reduced effective area is allowed for bearing resistance or settlement calculation. The point of load application shall be at the centroid of the reduced area.

LRFD 10.6.1.3

Note: As a practice, the average pressure and the values at the toe and heel under different load cases and limit states are provided to the MDOT Geotechnical Services Section for verification.

This example presents the LRFD and MDOT methods.

Strength I

Factored vertical force under dead load (DL)

$$F_{VFtDL} := 1.25 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \ldots + 1.5DW_{Sup} + 1.35 \cdot EV_{Ft}$$

$$F_{VFtDL} = 2.949 \times 10^{3} \cdot kip$$

Factored vertical force with live load $\quad F_{VFtStrI} = 3.687 \times 10^{3} \cdot kip \quad$ From Step 5.5

Factored moment about the longitudinal axis of the footing $\quad M_{XFtStrI} = 1.393 \times 10^{3} \cdot kip \cdot ft \quad$ From Step 5.5

Factored moment about the transverse axis of the footing $\quad M_{YFtStrI} = 894.546 \cdot kip \cdot ft \quad$ From Step 5.5

Eccentricity in the footing width direction

$$e_B := \frac{M_{XFtStrI}}{F_{VFtStrI}} = 0.378\ ft$$

Eccentricity in the footing length direction

$$e_L := \frac{M_{YFtStrI}}{F_{VFtStrI}} = 0.243\ ft$$

<u>LRFD Method</u>

Effective footing width $\quad B_{eff} := w_{footing} - 2 \cdot e_B = 9.244\ ft \quad$ **LRFD Eq. 10.6.1.3-1**

Effective footing length $\quad L_{eff} := l_{footing} - 2 \cdot e_L = 68.515\ ft \quad$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure

$$q_{bearing\_StrI} := \frac{F_{VFtStrI}}{B_{eff} \cdot L_{eff}} = 5.821 \cdot ksf$$

<u>MDOT Method</u>

Average bearing pressure under DL

$$q_{bearingDL\_StrI} := \frac{F_{VFtDL}}{w_{footing} \cdot l_{footing}} = 4.274 \cdot ksf$$

Average bearing pressure

$$q_{centerStrI} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} = 5.343 \cdot ksf$$

Maximum bearing pressure $q_{maxStrI} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} \left(1 + 6 \cdot \frac{e_B}{w_{footing}} + 6 \cdot \frac{e_L}{l_{footing}}\right) = 6.668 \cdot ksf$

Minimum bearing pressure $q_{minStrI} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} \left(1 - 6 \cdot \frac{e_B}{w_{footing}} - 6 \cdot \frac{e_L}{l_{footing}}\right) = 4.019 \cdot ksf$

Strength III

Factored vertical force $F_{VFtStrIII} = 2.949 \times 10^3 \cdot kip$ From Step 5.5

Factored moment about the longitudinal axis of the footing $M_{XFtStrIII} = 619.589 \cdot kip \cdot ft$ From Step 5.5

Factored moment about the transverse axis of the footing $M_{YFtStrIII} = 570.47 \cdot kip \cdot ft$ From Step 5.5

Eccentricity in the footing width direction $e_B := \frac{M_{XFtStrIII}}{F_{VFtStrIII}} = 0.21\ ft$

Eccentricity in the footing length direction $e_L := \frac{M_{YFtStrIII}}{F_{VFtStrIII}} = 0.193\ ft$

<u>LRFD Method</u>

Effective footing width $B_{eff} := w_{footing} - 2 \cdot e_B = 9.58\ ft$ **LRFD Eq. 10.6.1.3-1**

Effective footing length $L_{eff} := l_{footing} - 2 \cdot e_L = 68.613\ ft$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure $q_{bearing\_StrIII} := \frac{F_{VFtStrIII}}{B_{eff} \cdot L_{eff}} = 4.486 \cdot ksf$

<u>MDOT Method</u>

Average bearing pressure $q_{centerStrIII} := \frac{F_{VFtStrIII}}{w_{footing} \cdot l_{footing}} = 4.274 \cdot ksf$

Maximum bearing pressure $q_{maxStrIII} := \frac{F_{VFtStrIII}}{w_{footing} \cdot l_{footing}} \left(1 + 6 \cdot \frac{e_B}{w_{footing}} + 6 \cdot \frac{e_L}{l_{footing}}\right) = 4.884 \cdot ksf$

Minimum bearing pressure $q_{minStrIII} := \frac{F_{VFtStrIII}}{w_{footing} \cdot l_{footing}} \left(1 - 6 \cdot \frac{e_B}{w_{footing}} - 6 \cdot \frac{e_L}{l_{footing}}\right) = 3.663 \cdot ksf$

Strength V

Factored vertical force $F_{VFtStrV} = 3.518 \times 10^3 \cdot kip$ From Step 5.5

Factored moment about the longitudinal axis of the footing $M_{XFtStrV} = 1.694 \times 10^3 \cdot kip \cdot ft$ From Step 5.5

Factored moment about the transverse axis of the footing $M_{YFtStrV} = 1.324 \times 10^3 \cdot kip \cdot ft$ From Step 5.5

Eccentricity in the footing width direction $e_B := \dfrac{M_{XFtStrV}}{F_{VFtStrV}} = 0.481\ ft$

Eccentricity in the footing length direction $e_L := \dfrac{M_{YFtStrV}}{F_{VFtStrV}} = 0.376\ ft$

LRFD Method

Effective footing width $B_{eff} := w_{footing} - 2 \cdot e_B = 9.037\ ft$ **LRFD Eq. 10.6.1.3-1**

Effective footing length $L_{eff} := l_{footing} - 2 \cdot e_L = 68.247\ ft$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure $q_{bearing\_StrV} := \dfrac{F_{VFtStrV}}{B_{eff} \cdot L_{eff}} = 5.704 \cdot ksf$

MDOT Method

Average bearing pressure $q_{centerStrV} := \dfrac{F_{VFtStrV}}{w_{footing} \cdot l_{footing}} = 5.099 \cdot ksf$

Maximum bearing pressure $q_{maxStrV} := \dfrac{F_{VFtStrV}}{w_{footing} \cdot l_{footing}} \left(1 + 6 \cdot \dfrac{e_B}{w_{footing}} + 6 \cdot \dfrac{e_L}{l_{footing}}\right) = 6.738 \cdot ksf$

Minimum bearing pressure $q_{minStrV} := \dfrac{F_{VFtStrV}}{w_{footing} \cdot l_{footing}} \left(1 - 6 \cdot \dfrac{e_B}{w_{footing}} - 6 \cdot \dfrac{e_L}{l_{footing}}\right) = 3.459 \cdot ksf$

Service I

Factored vertical force under dead load (DL)

$$F_{VFtDLSerI} := DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing} \cdots + DW_{Sup} + EV_{Ft}$$

$$F_{VFtDLSerI} = 2.312 \times 10^3 \cdot kip$$

Factored vertical force with live load $F_{VFtSerI} = 2.734 \times 10^3 \cdot kip$ From Step 5.5

Factored moment about the longitudinal axis of the footing $M_{XFtSerI} = 1.316 \times 10^3 \cdot kip \cdot ft$ From Step 5.5

Factored moment about the transverse axis of the footing $M_{YFtSerI} = 1.141 \times 10^3 \cdot kip \cdot ft$ From Step 5.5

Eccentricity in the footing width direction $e_B := \dfrac{M_{XFtSerI}}{F_{VFtSerI}} = 0.481\ ft$

Eccentricity in the footing length direction $e_L := \dfrac{M_{YFtSerI}}{F_{VFtSerI}} = 0.417\ ft$

LRFD Method

Effective footing width $B_{eff} := w_{footing} - 2 \cdot e_B = 9.037\ ft$ **LRFD Eq. 10.6.1.3-1**

Effective footing length $L_{eff} := l_{footing} - 2 \cdot e_L = 68.165\ ft$ **LRFD Eq. 10.6.1.3-1**

Bearing pressure $$q_{bearing\_SerI} := \frac{F_{VFtSerI}}{B_{eff} \cdot L_{eff}} = 4.438 \cdot ksf$$

MDOT Method

Bearing pressure under DL $$q_{bearingDL\_SerI} := \frac{F_{VFtDLSerI}}{w_{footing} \cdot l_{footing}} = 3.351 \cdot ksf$$

Average bearing pressure $$q_{centerSerI} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} = 3.962 \cdot ksf$$

Maximum bearing pressure $$q_{maxSerI} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} \left(1 + 6 \cdot \frac{e_B}{w_{footing}} + 6 \cdot \frac{e_L}{l_{footing}}\right) = 5.251 \cdot ksf$$

Minimum bearing pressure $$q_{minSerI} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} \left(1 - 6 \cdot \frac{e_B}{w_{footing}} - 6 \cdot \frac{e_L}{l_{footing}}\right) = 2.674 \cdot ksf$$

Summary

LRFD Method

For LRFD method, the controlling bearing pressure under strength limit states is

$$q_b := \max\left(q_{bearing\_StrI}, q_{bearing\_StrIII}, q_{bearing\_StrV}\right) = 5.821 \cdot ksf$$

The controlling bearing pressure needs to be checked with the factored bearing resistance of the soil provided by the Geotechnical Services Section.

MDOT Method

A summary of bearing pressure values is shown in the following table:

	Average bearing pressure DL only (psf)	Average bearing pressure (psf)	Bearing pressure max. (psf)	Bearing pressure min. (psf)
Service	3351	3962	5251	2674
Strength	4274	5343	6738	4019
Allowable	Provided by Geotechnical Services Section			

The Geotechnical Services Section uses these values for the verification of bearing resistance and settlement limits. If the bearing pressure exceeds the bearing strength of the soil, the size of the footing needs to be increased. **BDM 7.03.02G**

Settlement Check

The Geotechnical Services Section uses the controlling bearing pressure from the service limit state to check if the total settlement of foundation is less than 1.5 in., the allowable limit. **BDM 7.03.02G 2b**

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

$$q_{b\_settlement} := q_{bearing\_SerI} = 4.438 \cdot ksf$$

The Geotechnical Services Section uses this controlling bearing pressure to calculate the foundation total settlement.

For the MDOT method, the bearing pressures under service limit state are provided to the Geotechnical Services 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 Design Example.*

Sliding Resistance Check

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. **LRFD 10.6.3.4**

The Geotechnical Services Section should provide a coefficient of sliding resistance (μ) for a design. MDOT typically uses a sliding resistance coefficient of 0.5 for cast-in-place concrete footings. Consult the Geotechnical Services Section 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:

- Minimum load factors are used for all vertical loads.
- Maximum load factors are used for the loads that contribute to horizontal sliding forces.
- Live load is excluded.
- Since DW is the future wearing surface load, it is excluded.

The sliding resistance provided by the passive earth pressure is included in the design. **BDM 7.03.02F**

Passive earth pressure coefficient provided by the Geotechnical Services Section $k_p := 3.3$

Passive earth pressure at the footing base $p_p := k_p \cdot \gamma_s \cdot \left(h_{soil} + t_{footing}\right) = 1.98 \cdot ksf$

Nominal passive resistance of soil $R_{ep} := \frac{1}{2} \cdot p_p \cdot \left(h_{soil} + t_{footing}\right) \cdot l_{footing} = 341.55 \cdot kip$

Resistance factor for passive resistance $\phi_{ep} := 0.5$ **BDM 7.03.02F, LRFD Table 10.5.5.5.2-1**

Resistance factor for shear resistance between soil and foundation $\phi_\tau := 0.8$ **BDM 7.03.02F, LRFD Table 10.5.5.5.2-1**

Strength I

Factored shear force parallel to the transverse axis of the footing

$$V_{LFtStrI} = 56.875 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the footing

$$V_{TFtStrI} = 0 \cdot kip$$

Factored sliding force (Demand)

$$V_{sliding} := \sqrt{V_{LFtStrI}^2 + V_{TFtStrI}^2} = 56.875 \cdot kip$$

Minimum vertical load

$$F_{VFtStrIMin} := 0.9 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) + 1.0 \cdot \left(EV_{Ft}\right) = 1.927 \times 10^3 \cdot kip$$

Sliding resistance (Capacity)

$$V_{resistance} := \phi_T \cdot \mu \cdot F_{VFtStrIMin} + \phi_{ep} \cdot R_{ep} = 941.419 \cdot kip$$

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

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

Strength III

Factored shear force parallel to the transverse axis of the footing

$$V_{LFtStrIII} = 34.941 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the footing

$$V_{TFtStrIII} = 24.087 \cdot kip$$

Factored sliding force (Demand)

$$V_{sliding} := \sqrt{V_{LFtStrIII}^2 + V_{TFtStrIII}^2} = 42.439 \cdot kip$$

Minimum vertical load

$$F_{VFtStrIIIMin} := 0.9 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) + 1.0 \cdot \left(EV_{Ft}\right) = 1.927 \times 10^3 \cdot kip$$

Sliding resistance (Capacity)

$$V_{resistance} := \phi_T \cdot \mu \cdot F_{VFtStrIIIMin} + \phi_{ep} \cdot R_{ep} = 941.419 \cdot kip$$

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

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

Strength V

Factored shear force parallel to the transverse axis of the footing

$$V_{LFtStrV} = 75.724 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the footing

$$V_{TFtStrV} = 26.44 \cdot kip$$

Factored sliding force (Demand)

$$V_{sliding} := \sqrt{V_{LFtStrV}^2 + V_{TFtStrV}^2} = 80.207 \cdot kip$$

Minimum vertical load

$$F_{VFtStrVMin} := 0.9 \cdot \left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) + 1.0 \cdot \left(EV_{Ft}\right) = 1.927 \times 10^3 \cdot kip$$

Sliding resistance (Capacity)

$$V_{resistance} := \phi_T \cdot \mu \cdot F_{VFtStrVMin} + \phi_{ep} \cdot R_{ep} = 941.419 \cdot kip$$

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 footing length direction is not of a concern. The following calculations present the evaluation of the eccentricity in the footing width direction:

Strength I

Minimum vertical force $F_{VFtStrIMin} = 1.927 \times 10^{3} \cdot kip$

Moment about the longitudinal axis of the footing $M_{XFtStrI} = 1.393 \times 10^{3}\ ft \cdot kip$

Eccentricity in the footing width direction measured from the centerline $e_B := \frac{M_{XFtStrI}}{F_{VFtStrIMin}} = 0.723\ ft$

1/6 of footing width $\frac{w_{footing}}{6} = 1.667\ ft$

Check if the load eccentricity limitation is satisfied $Check := if\left(e_B < \frac{w_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Strength III

Minimum vertical force $F_{VFtStrIIIMin} = 1.927 \times 10^{3} \cdot kip$

Moment about the longitudinal axis of the footing $M_{XFtStrIII} = 619.589\ ft \cdot kip$

Eccentricity in the footing width direction measured from the centerline $e_B := \frac{M_{XFtStrIII}}{F_{VFtStrIIIMin}} = 0.322\ ft$

Check if the load eccentricity limitation is satisfied $Check := if\left(e_B < \frac{w_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Strength V

Minimum vertical force $F_{VFtStrVMin} = 1.927 \times 10^{3} \cdot kip$

Moment about the longitudinal axis of the footing $M_{XFtStrV} = 1.694 \times 10^{3}\ ft \cdot kip$

Eccentricity in the footing width direction measured from the centerline $e_B := \frac{M_{XFtStrV}}{F_{VFtStrVMin}} = 0.879\ ft$

Check if the load eccentricity limitation is satisfied $Check := if\left(e_B < \frac{w_{footing}}{6}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$

Step 5.10 Structural Design of the Footing

Description

This step presents the structural design of the pier footing.

Page Contents

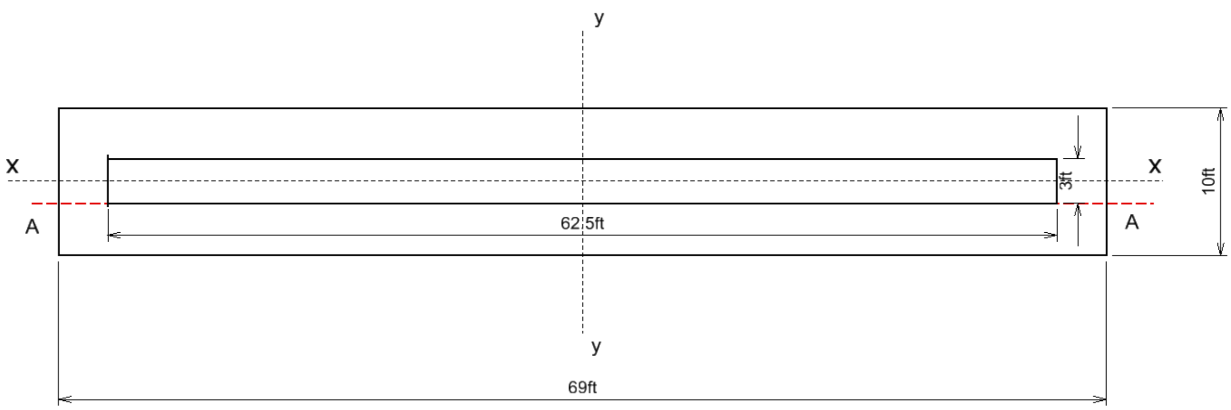
For structural design of an eccentrically loaded foundation, a triangular or trapezoidal bearing pressure distribution shall be used.

LRFD 10.6.5

Design for Flexure

Transverse Reinforcement

The critical section A-A for the design of transverse flexural reinforcement is located at the face of the base wall, as shown in the following figure. In the absence of a base wall, the critical section is located at the face of a column.



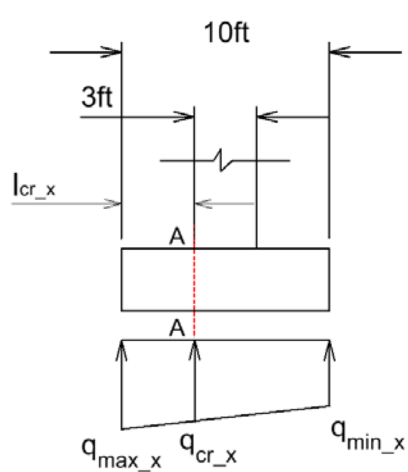
Distance from the edge of footing to the face of the base wall $\quad l_{cr\_x} := \dfrac{w_{footing} - t_{wall}}{2} = 3.5\ ft$

Section modulus of the footing about x-axis $\quad S_{XFt} := \dfrac{1}{6} l_{footing} \cdot w_{footing}^{\ 2} = 1.15 \times 10^{3} \cdot ft^{3}$

As per the combined load effects presented in Step 5.5, the Strength I limit state is the governing case for flexural design.

Factored vertical force $\quad F_{VFtStrI} = 3.687 \times 10^{3} \cdot kip$

Factored moment about x-axis $\quad M_{XFtStrI} = 1.393 \times 10^{3} \cdot kip \cdot ft$



Maximum and minimum bearing pressure

$$q_{max\_x} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} + \frac{M_{XFtStrI}}{S_{XFt}} = 6.555 \cdot ksf$$

$$q_{min\_x} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} - \frac{M_{XFtStrI}}{S_{XFt}} = 4.131 \cdot ksf$$

Bearing pressure at the critical section

$$q_{cr\_x} := q_{min\_x} + \frac{\left(q_{max\_x} - q_{min\_x}\right)}{w_{footing}} \cdot \left(w_{footing} - l_{cr\_x}\right) = 5.707 \cdot ksf$$

This example uses a simplified analysis method to determine the maximum moments at the face of the wall by selecting load factors to produce the maximum bearing 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 resisting forces (such as the overburden pressure and footing self-weight) to calculate the maximum moment at the face of the wall.

The moment demand at the critical section on a per-foot basis:

$$M_{ux} := q_{cr\_x} \cdot \frac{{l_{cr\_x}}^2}{2} + \left(q_{max\_x} - q_{cr\_x}\right) \cdot \frac{{l_{cr\_x}}^2}{3} - 0.9 \cdot W_c \cdot t_{footing} \cdot \frac{{l_{cr\_x}}^2}{2} - 1.0 \gamma_s \cdot h_{soil} \cdot \frac{{l_{cr\_x}}^2}{2}$$

$$M_{ux} = 34.466 \cdot \frac{kip \cdot ft}{ft}$$

<u>Flexural Resistance</u> **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.

Select a trial bar size $bar := 6$

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

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

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**
12 in. for walls and footings greater than 18 in.
For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. in base walls and pier footings adjacent to roadways. **BDG 5.22.01**

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

Select a spacing for reinforcing steel bars $s_{bar} := 12 \cdot in$

Select a 1-ft wide strip for the design.

Area of tension steel in a 1-ft wide strip

$$A_{sProvide\_x} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.44 \cdot in^2$$

Since the moment about the x-axis is greater than the moment about the y-axis, place the reinforcing bars along the footing width direction at the bottom of the footing. The reinforcing bars along the footing length direction should be placed directly on top of the bars along the width direction.

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

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

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

Stress block factor $\beta_1 = 0.85$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

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

Required area of steel $A_{sRequired\_x} := Find(A_s) = 0.241 \cdot in^2$

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

Moment capacity of the section with the provided steel

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

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

Distance from the extreme compression fiber to the neutral axis $c := \frac{A_{sProvide\_x} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.01 \cdot in$

Check the validity of 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 be adequate to develop a factored flexural resistance at least 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 ASTM A615 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 $1.33 \cdot M_{ux} = 45.839 \cdot \frac{kip \cdot ft}{ft}$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33M_{ux}, M_{cr}\right) = 45.839 \cdot \frac{kip \cdot ft}{ft}$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

The spacing requirement 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest flexural reinforcement $d_c := Cover_{ft} = 4 \cdot in$

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 cross-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 $x := 5 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvide\_x} \cdot \left(d_{ex} - x\right)$

Position of the neutral axis $x_{na} := Find(x) = 4.049 \cdot in$

Vertical force and moment at the base of the footing under the Service I limit state

$F_{VFtSerI} = 2.734 \times 10^3 \cdot kip$ $M_{XFtSerI} = 1.316 \times 10^3 \cdot kip \cdot ft$

Maximum and minimum bearing pressure $q_{max} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} + \frac{M_{XFtSerI}}{S_{XFt}} = 5.107 \cdot ksf$

$q_{min} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} - \frac{M_{XFtSerI}}{S_{XFt}} = 2.818 \cdot ksf$

Bearing pressure at the critical section under the Service I limit state

$$q_{crSerI} := q_{min} + \frac{(q_{max} - q_{min})}{w_{footing}} \cdot (w_{footing} - l_{cr\_x}) = 4.306 \cdot ksf$$

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

$$M_{rSerI\_x} := q_{crSerI} \cdot \frac{l_{cr\_x}^{\,2}}{2} + (q_{max} - q_{crSerI}) \cdot \frac{l_{cr\_x}^{\,2}}{3} - W_c \cdot t_{footing} \cdot \frac{l_{cr\_x}^{\,2}}{2} - \gamma_s \cdot h_{soil} \cdot \frac{l_{cr\_x}^{\,2}}{2}$$

$$M_{rSerI\_x} = 25.417 \cdot \frac{kip \cdot ft}{ft}$$

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

$$T_s := \frac{M_{rSerI\_x}}{d_{ex} - \frac{x_{na}}{3}} \cdot ft = 10 \cdot kip$$

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

$$f_{ss1} := \frac{T_s}{A_{sProvide\_x}} = 22.616 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min(f_{ss1}, 0.6f_y) = 22.616 \cdot ksi$$

Required reinforcement spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 18.262 \cdot in$$

Check if the spacing provided < the required spacing

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

Shrinkage and Temperature Reinforcement Requirement

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the footing: **LRFD 5.10.6**

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} := \min\left[\begin{bmatrix} \left(0.60 \frac{in^2}{ft}\right) \\ \max\left[\begin{bmatrix} \left(0.11 \frac{in^2}{ft}\right) \\ \left[\frac{1.3 \cdot w_{footing} \cdot t_{footing} \cdot \frac{kip}{in \cdot ft}}{2(w_{footing} + t_{footing}) \cdot f_y}\right] \end{bmatrix}\right] \end{bmatrix}\right] \cdot ft = 0.3 \cdot in^2$$

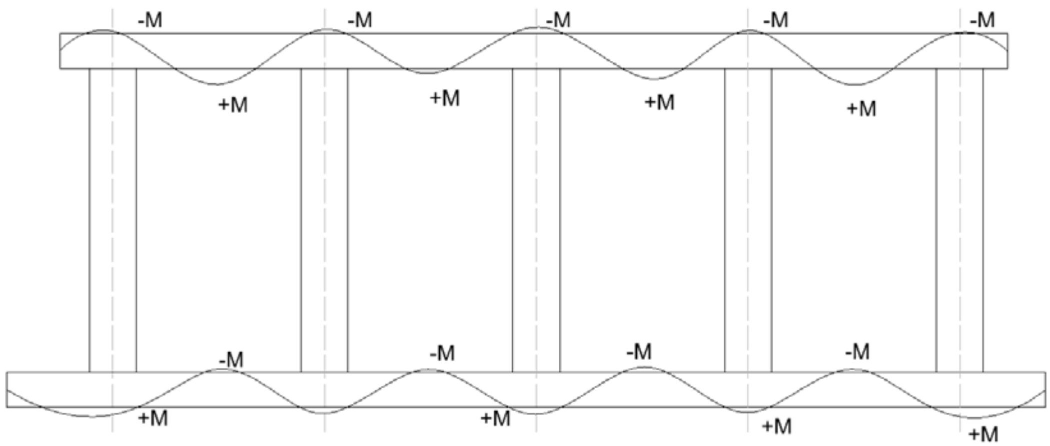
Check if the provided area of steel > the required area of shrinkage and temperature steel

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

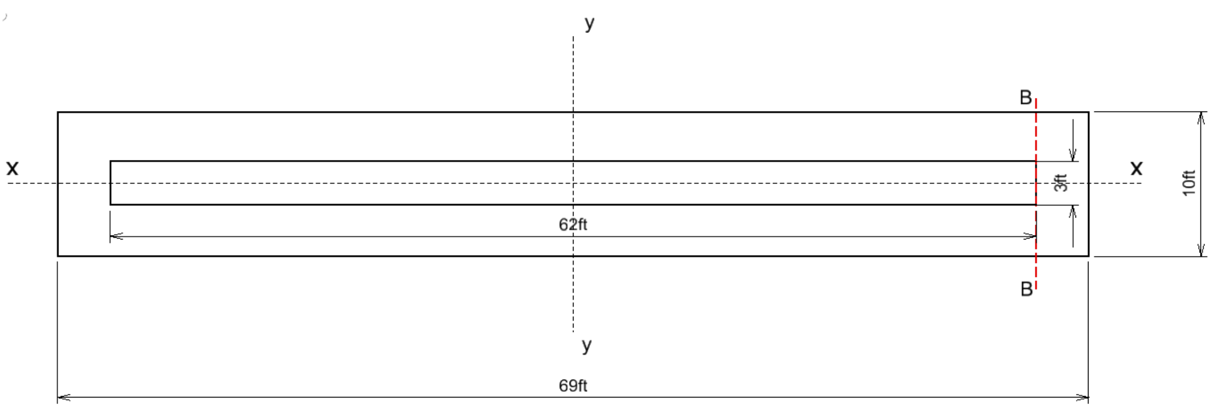
Therefore, the flexural design requires the use of No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the transverse flexural reinforcement at the bottom of the footing.

Longitudinal Reinforcement

For flexural design of the reinforcement along the footing length direction, the footing with the base wall is modeled as a multi-span continuous beam supported at the columns. The critical section for the positive moment is located at the face of the base wall. The critical section for the negative moment is located towards the center of the span between two neighboring columns.



Positive Moment at the Face of the Base Wall (Section B-B)

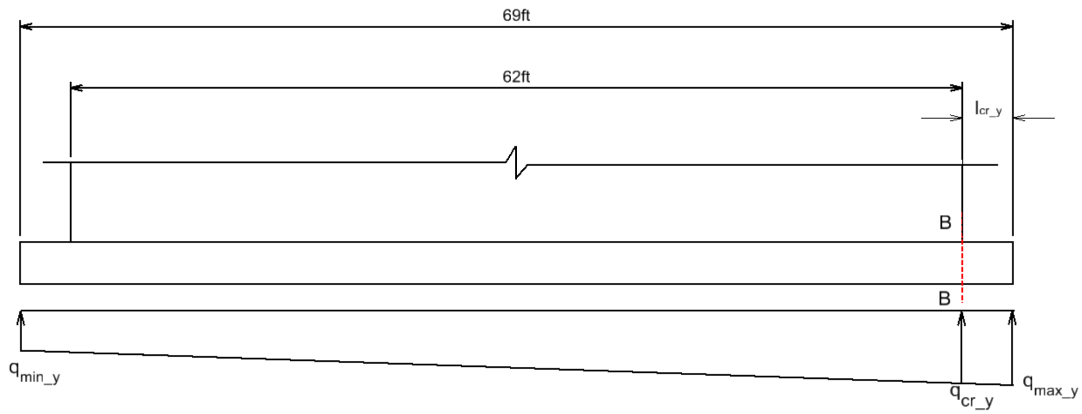


Section modulus of the footing about y-axis $S_{YFt} := \frac{1}{6} w_{footing} \cdot l_{footing}^{2} = 7.935 \times 10^{3} \cdot ft^{3}$

As per the combined load effects presented in Step 5.5, the Strength I limit state is the governing case for flexural design.

Factored vertical force $F_{VFtStrI} = 3.687 \times 10^{3} \cdot kip$

Factored moment about y-axis $M_{YFtStrI} = 894.546 \cdot kip \cdot ft$



Maximum and minimum bearing pressure

$$q_{max\_y} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} + \frac{M_{YFtStrI}}{S_{YFt}} = 5.456 \cdot ksf$$

$$q_{min\_y} := \frac{F_{VFtStrI}}{w_{footing} \cdot l_{footing}} - \frac{M_{YFtStrI}}{S_{YFt}} = 5.23 \cdot ksf$$

Distance from the edge of the footing to the critical section

$$l_{cr\_y} := \frac{l_{footing} - l_{wall}}{2} = 3.5\ ft$$

Bearing pressure at the critical section

$$q_{cr\_y} := q_{min\_y} + \frac{\left(q_{max\_y} - q_{min\_y}\right)}{l_{footing}} \cdot \left(l_{footing} - l_{cr\_y}\right) = 5.444 \cdot ksf$$

This example uses a simplified analysis method to determine the maximum moments at the face of the wall by selecting load factors to produce the maximum bearing 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 resisting forces (such as the overburden pressure and footing self-weight) to calculate the maximum moment at the critical section.

The moment demand at the critical section on a per-foot basis

$$M_{uy} := q_{cr\_y} \cdot \frac{l_{cr\_y}^{\ 2}}{2} + \left(q_{max\_y} - q_{cr\_y}\right) \cdot \frac{l_{cr\_y}^{\ 2}}{3} - 0.9 \cdot W_c \cdot t_{footing} \cdot \frac{l_{cr\_y}^{\ 2}}{2} - 1.0 \gamma_s \cdot h_{soil} \cdot \frac{l_{cr\_y}^{\ 2}}{2}$$

$$M_{uy} = 29.443 \cdot \frac{kip \cdot ft}{ft}$$

Flexural 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.

Select a trial bar size $bar := 6$

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

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

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. in base walls and pier footings adjacent to roadways. **BDG 5.22.01**

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

Select a spacing for reinforcing steel bars $s_{bar} := 12\cdot in$

Select a 1-ft wide strip for the design.

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

Since the moment about the y-axis is smaller than the moment about the x-axis, place the reinforcing bars along the width direction at the bottom of the footing. The reinforcing bars along the length direction should be placed directly on top of the bars along width direction.

Effective depth $d_{ey} := t_{footing} - Cover_{ft} - \dfrac{d_{bx}}{2} - \dfrac{d_{by}}{2} = 31.25\cdot in$

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

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

Stress block factor $\beta_1 = 0.85$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

Given $M_{uy}\cdot ft = \phi_f\cdot A_s\cdot f_y\cdot\left[d_{ey} - \dfrac{1}{2}\cdot\left(\dfrac{A_s\cdot f_y}{0.85\cdot f_c\cdot b}\right)\right]$

Required area of steel $A_{sRequired\_y} := Find(A_s) = 0.211\cdot in^2$

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

Moment capacity of the section with the provided steel

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

$$M_{Provided} = 61.021\cdot\frac{kip\cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis $c := \dfrac{A_{sProvided\_y}\cdot f_y}{0.85\cdot f_c\cdot\beta_1\cdot b} = 1.01\cdot in$

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

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must be adequate to develop a factored flexural resistance at least 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 ASTM A615 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 $1.33 \cdot M_{uy} = 39.159 \cdot \frac{kip \cdot ft}{ft}$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33M_{uy}, M_{cr}\right) = 39.159 \cdot \frac{kip \cdot ft}{ft}$

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

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

The spacing requirement 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest flexural reinforcement $d_c := Cover_{ft} = 4 \cdot in$

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 cross-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 $x := 5 \cdot in$

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

Position of the neutral axis $x_{na} := Find(x) = 3.998 \cdot in$

Vertical force and moment at the base of the footing under the Service I limit state

$$F_{VFtSerI} = 2.734 \times 10^3 \cdot kip \qquad M_{YFtSerI} = 1.141 \times 10^3 \cdot kip \cdot ft$$

Maximum and minimum bearing pressure

$$q_{max} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} + \frac{M_{YFtSerI}}{S_{YFt}} = 4.106 \cdot ksf$$

$$q_{min} := \frac{F_{VFtSerI}}{w_{footing} \cdot l_{footing}} - \frac{M_{YFtSerI}}{S_{YFt}} = 3.819 \cdot ksf$$

Bearing pressure at the critical section under the Service I limit state

$$q_{crSerI} := q_{min} + \frac{(q_{max} - q_{min})}{l_{footing}} \cdot (l_{footing} - l_{cr\_y}) = 4.092 \cdot ksf$$

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

$$M_{rSerI\_y} := q_{crSerI} \cdot \frac{l_{cr\_y}^2}{2} + (q_{max} - q_{crSerI}) \cdot \frac{l_{cr\_y}^2}{3} - W_c \cdot t_{footing} \cdot \frac{l_{cr\_y}^2}{2} - \gamma_s \cdot h_{soil} \cdot \frac{l_{cr\_y}^2}{2}$$

$$M_{rSerI\_y} = 20.894 \cdot \frac{kip \cdot ft}{ft}$$

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

$$T_s := \frac{M_{rSerI\_y}}{d_{ey} - \frac{x_{na}}{3}} \cdot ft = 8.4 \cdot kip$$

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

$$f_{ss1} := \frac{T_s}{A_{sProvided\_y}} = 19.047 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min(f_{ss1}, 0.6f_y) = 19.047 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 23.183 \cdot in$$

Check if the spacing provided < the required spacing

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

Shrinkage and Temperature Reinforcement

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the footing: **LRFD 5.10.6**

Minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} = 0.3 \cdot in^2$$

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

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

Therefore, the flexural design requires the use of No. 6 bars at 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the longitudinal flexural reinforcement at the bottom of the footing.

Negative Moment Design

The maximum negative moment within the span is located between two neighboring columns. The footing is modeled as a continuous beam supported at the columns. The Strength I limit state developed the maximum bearing pressure. The top flexural reinforcement required at the maximum negative moment section is designed using the procedure implemented for the design of bottom longitudinal steel at section B-B.

Note: Considering MDOT practice, the contribution of compression steel towards the flexural strength is neglected.

A Footing With a Base Wall

The pier is designed with a 6-ft tall base wall. The negative moment results in tension at the top of the base wall.

Note: A section of spread footing with a base wall behaves as an inverted T-beam. As per the MDOT practice, the base wall is conservatively designed as a rectangular section excluding the contribution of the footing.

Strength I is the governing limit state for the negative moment design.

Maximum negative moment within the bay, Strength I (from structural analysis) $M_{wall\_neg} := 507.8\,kip \cdot ft$

Maximum negative moment within the bay, Service I (from structural analysis) $M_{wall\_neg\_SerI} := 349.7\,kip \cdot ft$

Flexural 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.

Effective depth $d_e := h_{wall} + t_{footing} - Cover_{walltop} = 105 \cdot in$

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

Width of the compression face of the member $b := t_{wall} = 3\,ft$

It is assumed that there will be only one layer of negative moment reinforcement.

Select a trial bar size $bar := 5$

Nominal diameter of a reinforcing steel bar $d_{bar} := Dia(bar) = 0.625 \cdot in$

Cross-section area of the bar $A_{bar} := Area(bar) = 0.31 \cdot in^2$

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

Initial assumption $A_s := 1in^2$

Given $$M_{wall\_neg} = \phi_f \cdot A_s \cdot f_y \cdot \left[d_e - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b} \right) \right]$$ **LRFD 5.6.3.2**

Required area of steel $A_{sRequired\_neg} := Find(A_s) = 1.078 \cdot in^2$

Required number of rebars $$n_{bar\_neg.req} := \frac{A_{sRequired\_neg}}{A_{bar}} = 3.478$$

Number of reinforcing steel bars provided $n_{bar\_neg.provided} := 4$

Amount of steel provided in the section $A_{sProvided\_neg} := n_{bar\_neg.provided} \cdot A_{bar} = 1.24 \cdot in^2$

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

Moment capacity of the section with the provided steel

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

$$M_{Capacity\_neg} = 583.639 \cdot kip \cdot ft$$

Distance from the extreme compression fiber to the neutral axis

$$c := \frac{A_{sProvided\_neg} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 0.95 \cdot in$$

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 be adequate to develop a factored flexural resistance at least 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 ASTM A615 Grade 60 reinforcement

Section modulus $S_c := \frac{1}{6} \cdot b \cdot h_{cap}^2 = 1.058 \times 10^4 \cdot in^3$

Cracking moment $M_{cr} := \gamma_3 \cdot \gamma_1 \cdot f_r \cdot S_c = 393.039 \cdot kip \cdot ft$

1.33 times the factored moment demand $1.33 \cdot M_{uNStrI} = 905.331 \cdot kip \cdot ft$

The factored moment to satisfy the minimum reinforcement requirement $M_{req} := \min\left(1.33 M_{uNStrI}, M_{cr}\right) = 393.039 \cdot kip \cdot ft$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

Spacing requirement 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from extreme tension fiber to the center of the closest flexural reinforcement

$$d_c := Cover_{wall} = 3.75 \cdot in$$

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

$$\beta_s := 1 + \frac{d_c}{0.7\left(h_{cap} - d_c\right)} = 1.14$$

The position of the cross-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

$$x := 6in$$

Given

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

Position of the neutral axis

$$x_{na} := Find(x) = 7.336 \cdot in$$

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

$$T_s := \frac{M_{wall\_neg\_SerI}}{d_e - \frac{x_{na}}{3}} = 40.9 \cdot kip$$

Stress in the steel reinforcement due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided\_neg}} = 32.999 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 32.999 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 11.107 \cdot in$$

Number of spaces between steel reinforcing bars provided at the top of the base wall

$$N_{spacingBWT} := n_{bar\_neg.provided} - 1 = 3$$

Spacing between steel reinforcing bars at the top of the base wall

$$s_{bar} := \frac{\left(t_{wall} - 2 \cdot Cover_{wall}\right)}{N_{spacingBWT}} = 9.5 \cdot in$$

Check if the spacing provided < the required spacing

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

Therefore, the flexural design requires 4 No. 5 bars ($A_s = 1.24$ in.^{2}) at the top of the base wall.

Design for Skin Reinforcement

LRFD 5.6.7

Distance from the extreme compression fiber to the centroid of the extreme tension steel

$$d_l := h_{wall} + t_{footing} - Cover_{walltop} = 105 \cdot in$$

The maximum spacing of skin reinforcement shall not exceed the lesser of $\frac{d_l}{6} = 17.5 \cdot in$ and 12 in.

Required area of skin reinforcement in in.^{2}/ft of height on each side face of the pier cap

$$A_{sk1} := 0.012 \cdot \left(d_l - 30in\right) \cdot \frac{in}{ft} = 0.9 \cdot \frac{in^2}{ft}$$

LRFD Eq. 5.6.7-3

Area of skin reinforcement need not exceed one fourth of the required flexural tensile reinforcement

$$A_{sk2} := \frac{1}{4} \cdot \frac{A_{sRequired\_neg}}{0.5 d_l} = 0.062 \cdot \frac{in^2}{ft}$$

$$A_{sk\_required} := \min\left(A_{sk1}, A_{sk2}\right) = 0.062 \cdot \frac{in^2}{ft}$$

Note: The base wall design presented in Step 5.8, requires No. 4 bars at 12 in. spacing as the horizontal shrinkage and temperature steel reinforcement. It is necessary to check if the already provided steel is adequate to satisfy the skin reinforcement requirement.

Therefore, Bar size $bar := 4$

Bar spacing $s := 12in$

Cross-section area of a reinforcing steel bar $A_{bar} := Area(bar) = 0.2 \cdot in^2$

Area of skin reinforcement provided on each side face of the base wall

$$A_{sk\_provided} := A_{bar} \cdot \frac{\frac{12in}{s}}{ft} = 0.2 \cdot \frac{in^2}{ft}$$

Check if $A_{sk\_provided} > A_{sk\_required}$

$$Check := if\left(A_{sk\_provided} > A_{sk\_required}, "OK", "Not OK"\right) = "OK"$$

Footing without a Base Wall

In the absence of a base wall, an adequate amount of longitudinal steel needs to be provided at the top of the footing to resist the moment. For illustrative purposes, the negative moment design of a footing without a base wall is presented.

Select a 1-ft wide strip for the design.

Maximum negative moment acting on a 1-ft wide strip, Strength I (from structural analysis)

$$M_{u\_neg} := 53.48 \frac{kip \cdot ft}{ft}$$

Maximum negative moment acting on a 1-ft wide strip, Service I (from structural analysis)

$$M_{u\_neg\_SerI} := 37.8 \frac{kip \cdot ft}{ft}$$

<u>Flexural Resistance</u> **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.

Select a trial bar size $bar := 7$

Nominal diameter of a reinforcing steel bar $d_{bar} := Dia(bar) = 0.875 \cdot in$

Cross-section area of a bar on the flexural tension side $A_{bar} := Area(bar) = 0.6 \cdot in^2$

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. in base walls and pier footings adjacent to roadways. **BDG 5.22.01**

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

Select a spacing for reinforcing steel bars $s_{bar} := 12 \cdot in$

Area of tension steel in a 1-ft wide strip $A_{sProvided\_neg} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.6 \cdot in^2$

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

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

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

Stress block factor $\beta_1 = 0.85$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

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

Required area of steel $A_{sRequired\_neg} := Find(A_s) = 0.376 \cdot in^2$

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

Moment capacity of the section with the provided steel

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

$$M_{Provided} = 82.787 \cdot \frac{kip \cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis $c := \frac{A_{sProvided\_neg} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.38 \cdot in$

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

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must be adequate to develop a factored flexural resistance at least 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 ASTM A615 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

$$1.33 \cdot M_{u\_neg} = 71.128 \cdot \frac{kip \cdot ft}{ft}$$

The factored moment to satisfy the minimum reinforcement requirement

$$M_{req} := \min\left(1.33 M_{u\_neg}, M_{cr}\right) = 71.128 \cdot \frac{kip \cdot ft}{ft}$$

Check the adequacy of the 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

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. Checking for crack control assures that the actual stress in the reinforcement does not exceed the service limit state stress.

Spacing requirement 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 the Class 1 exposure condition

$$\gamma_e := 1.00$$

Distance from extreme tension fiber to the center of the closest flexural reinforcement

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

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer closer 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 cross-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

$$x := 5 \cdot in$$

Given

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

Position of the neutral axis

$$x_{na} := Find(x) = 4.675 \cdot in$$

Maximum negative moment acting on a 1-ft wide strip, Service I (from structural analysis)

$$M_{u\_neg\_SerI} = 37.8 \cdot \frac{kip \cdot ft}{ft}$$

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

$$T_s := \frac{M_{u\_neg\_SerI}}{d_{ey} - \frac{x_{na}}{3}} \cdot ft = 15.3 \cdot kip$$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided\_neg}} = 25.462 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 25.462 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 15.327 \cdot in$$

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

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

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

Design for Shear

One-Way Shear at a Section Parallel to the Transverse Axis of the Footing

The factored shear force at the critical section is the resultant force due to the bearing pressure acting on the footing base area located outside of the critical section.

Note: Since the transverse and longitudinal load effects are considered independent, bearing pressure distribution across the footing width is uniform. Therefore, a 1-ft wide strip is considered for the design.

Effective width of the section

$$b = 12 \cdot in$$

Depth of an equivalent rectangular stress block

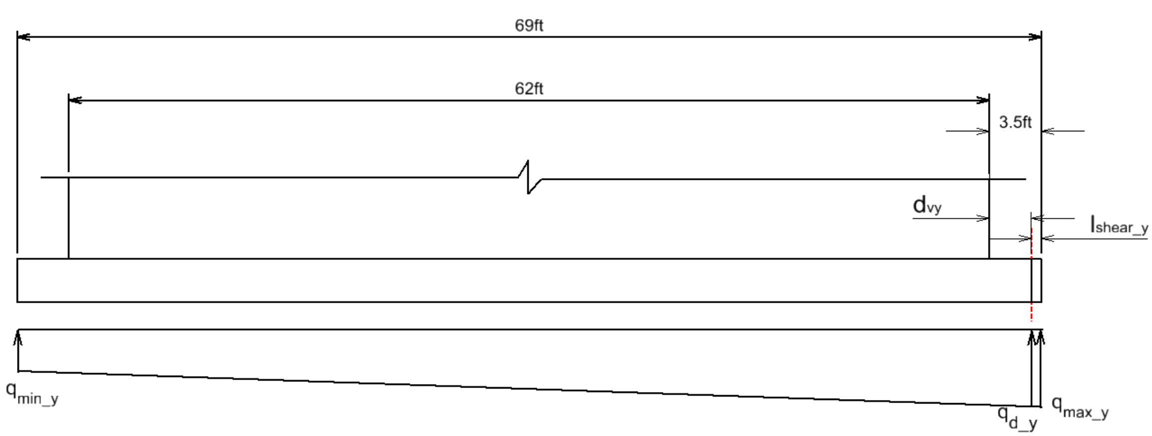
$$a := \frac{A_{sProvided\_y} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.863 \cdot in$$

Effective shear depth

$$d_{vy} := \max\left(d_{ey} - \frac{a}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot t_{footing}\right) = 30.819 \cdot in$$

LRFD 5.7.2.8

As shown in the following figure, the critical section for shear is located at a distance d_{vy} from the face of the base wall :



Distance from end of the footing to the critical section for shear

$$l_{shear\_y} := l_{cr\_y} - d_{vy} = 0.932 \cdot ft$$

Bearing stress at the critical section for shear

$$q_{d\_y} := q_{min\_y} + \frac{(q_{max\_y} - q_{min\_y})}{w_{footing}} \cdot (w_{footing} - l_{shear\_y}) = 5.435 \cdot ksf$$

Minimum load factors are used for the resisting forces (such as the overburden pressure and footing self-weight) to calculate the maximum shear at the critical section.

Factored shear demand at the critical section

$$V_{uFt\_y} := \frac{(q_{max\_y} + q_{d\_y})}{2} \cdot l_{shear\_y} - 0.9 \cdot W_c \cdot t_{footing} \cdot l_{shear\_y} - 1.0 \cdot \gamma_s \cdot h_{soil} \cdot l_{shear\_y} = 4.473 \cdot \frac{kip}{ft}$$

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

$$Check := if(l_{cr\_y} < 3 \cdot d_{vy}, \text{"Use the simplied method"}, \text{"Do not use the simplifed method"}) = \text{"Use the simplied method"}$$

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

$$\beta := 2$$

The nominal shear resistance of concrete, V_n, is calculated as follows:

$$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_{vy} = 40.5 \cdot kip$$ **LRFD Eq. 5.7.3.3-3**

$$V_{c2} := 0.25 f_c \cdot b \cdot d_{vy} = 277.368 \cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := min(V_{c1}, V_{c2}) = 40.483 \cdot kip$$

Resistance factor for shear

$$\phi_v := 0.9$$ **LRFD 5.5.4.2**

Factored shear resistance (Capacity)

$$V_r := \phi_v \cdot V_n = 36.435 \cdot kip$$

Check if the shear capacity > the shear demand

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

One-Way Shear at a Section Parallel to the Longitudinal Axis of the Footing

The factored shear force at the critical section is the resultant force due to the bearing pressure acting on the footing base area located outside the critical section.

Note: Since the transverse and longitudinal load effects are considered independent, bearing pressure distribution across the footing length is uniform. Therefore, a 1-ft wide strip is considered for the design.

Effective width of the section

$$b = 12 \cdot in$$

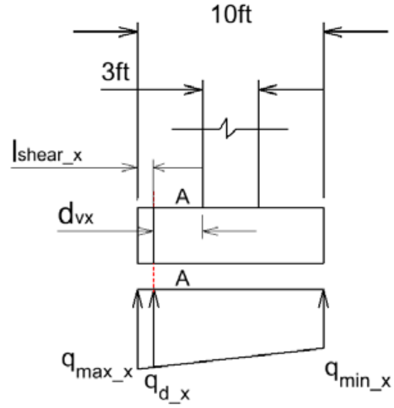
Depth of an equivalent rectangular stress block

$$a := \frac{A_{sProvide\_x} \cdot f_y}{0.85 \cdot f_c \cdot b} = 0.863 \cdot in$$

Effective shear depth

$$d_{vx} := max\left(d_{ex} - \frac{a}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot t_{footing}\right) = 31.569 \cdot in$$ **LRFD 5.7.2.8**

As shown in the following figure, the critical section for shear is located at a distance d_{vx} from the front face of the base wall :



Distance from end of the footing to the critical section for shear

$$l_{shear\_x} := l_{cr\_x} - d_{vx} = 0.869 \cdot ft$$

Bearing stress at the critical section for shear

$$q_{d\_x} := q_{min\_x} + \frac{(q_{max\_x} - q_{min\_x})}{w_{footing}} \cdot (w_{footing} - l_{shear\_x}) = 6.344 \cdot ksf$$

Minimum load factors are used for the resisting forces (such as the overburden pressure and footing self-weight) to calculate the maximum shear at the critical section.

Factored shear demand at the critical section

$$V_{uFt\_x} := \frac{(q_{max\_x} + q_{d\_x})}{2} \cdot l_{shear\_x} - 0.9 \cdot W_c \cdot t_{footing} \cdot l_{shear\_x} - 1.0 \cdot \gamma_s \cdot h_{soil} \cdot l_{shear\_x} = 5.046 \cdot \frac{kip}{ft}$$

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

$$Check := if(l_{cr\_x} < 3 \cdot d_{vx}, \text{"Use the simplied method"}, \text{"Do not use the simplifed method"}) = \text{"Use the simplied method"}$$

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:

$$V_{c1} := 0.0316 \cdot \beta \cdot \sqrt{f_c \cdot ksi} \cdot b \cdot d_{vx} = 41.5 \cdot kip$$ **LRFD Eq. 5.7.3.3-3**

$$V_{c2} := 0.25 f_c \cdot b \cdot d_{vx} = 284.118 \cdot kip$$ **LRFD Eq. 5.7.3.3-2**

$$V_n := min(V_{c1}, V_{c2}) = 41.468 \cdot kip$$

Resistance factor for shear

$$\phi_v := 0.9$$ **LRFD 5.5.4.2**

Factored shear resistance (Capacity)

$$V_r := \phi_v \cdot V_n = 37.321 \cdot kip$$

Check if the shear capacity > the shear demand

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

Two-Way Shear at a Critical Perimeter around the Base Wall

Two-way shear (punching shear) in the footing is checked at a critical perimeter around the base wall. In the absence of a base wall, two-way shear is checked at a critical perimeter around the columns.

The critical perimeter around the base wall, b_o, is located at a minimum of $0.5d_v$ from the perimeter of the base wall. **LRFD 5.12.8.6.3**

An average effective shear depth, d_v, should be used since the two-way shear area includes both x- and y-directions of the footing.

Average effective shear depth $$d_{v\_avg} := \frac{(d_{vx} + d_{vy})}{2} = 2.599\ \text{ft}$$

Critical perimeter $$b_0 := 2\cdot(l_{wall} + d_{v\_avg}) + 2\cdot(t_{wall} + d_{v\_avg}) = 140.398\ \text{ft}$$

Ratio of long to short side of critical perimeter $$\beta_c := \frac{l_{wall}}{t_{wall}} = 20.667$$

Nominal shear resistance $$V_{n1\_2way} := \left(0.063 + \frac{0.126}{\beta_c}\right)\cdot\sqrt{f_c\cdot ksi}\cdot b_0\cdot d_{v\_avg} = 6.29\times10^3\cdot kip$$

$$V_{n2\_2way} := 0.126\cdot\left(\sqrt{f_c\cdot ksi}\cdot b_0\cdot d_{v\_avg}\right) = 1.147\times10^4\cdot kip$$

LRFD Eq. 5.12.8.6.3-1

$$V_{n\_2way} := \min(V_{n1\_2way}, V_{n2\_2way}) = 6.29\times10^3\cdot kip$$

Factored shear resistance (Capacity) $$V_{r\_2way} := \phi_v\cdot V_{n\_2way} = 5.661\times10^3\cdot kip$$

To calculate the shear force acting on the critical perimeter, the average bearing pressure is used. The Strength I is the governing limit state.

Average bearing pressure $$q_{average} := \frac{F_{VFtStrI}}{w_{footing}\cdot l_{footing}} = 5.343\cdot\frac{kip}{ft^2}$$

Resultant shear force acting on the area outside of the critical perimeter (Demand)

$$V_{u\_2way} := q_{average}\cdot\left[w_{footing}\cdot l_{footing} - (l_{wall} + d_{v\_avg})\cdot(t_{wall} + d_{v\_avg})\right] = 1.754\times10^3\cdot kip$$

Check if the factored two-way shear resistance > the demand $$Check := if(V_{r\_2way} > V_{u\_2way}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$$

Two-Way Shear in a Footing without a Base Wall

Although the pier in this design example has a base wall, for illustrative purposes, the two-way shear check around the perimeter of a column is described below.

The critical perimeter of a column, b_o, is located at a minimum of $0.5d_v$ from the perimeter of the column. If portions of the critical perimeter are located off the footing, the critical perimeter is limited by the footing edge. **LRFD 5.12.8.6.3**

Critical perimeter $b_0 := \pi \cdot (d_{col} + d_{v\_avg}) = 16.02\,ft$

Ratio of long to short side of critical perimeter for circular column $\beta_c := 1$

Nominal shear resistance

$$V_{n1\_2way} := \left(0.063 + \frac{0.126}{\beta_c}\right) \cdot \sqrt{f_c \cdot ksi} \cdot b_0 \cdot d_{v\_avg} = 1.963 \times 10^3 \cdot kip$$

$$V_{n2\_2way} := 0.126 \cdot \left(\sqrt{f_c \cdot ksi} \cdot b_0 \cdot d_{v\_avg}\right) = 1.309 \times 10^3 \cdot kip$$

LRFD Eq. 5.12.8.6.3-1

$$V_{n\_2way} := \min\left(V_{n1\_2way}, V_{n2\_2way}\right) = 1.309 \times 10^3 \cdot kip$$

Factored shear resistance (Capacity) $V_{r\_2way} := \phi_v \cdot V_{n\_2way} = 1.178 \times 10^3 \cdot kip$

As shown in Step 5.5, the Strength I limit state yielded the maximum factored axial force in the column.

$$P_{uMax} := P_{u\_StrI} = 862.6 \cdot kip$$

Check if the factored two-way shear resistance > the demand

$$Check := if\left(V_{r\_2way} > P_{uMax}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

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**

Longitudinal Direction of the Footing

Available development length

$$l_{dy\_avail} := \frac{l_{footing} - l_{wall}}{2} - Cover_{ft} = 38 \cdot in$$

Assuming that the bars are at high stress, the required development length for No. 6 bars at 12 in. spacing

$$l_{dy.req} := 15in$$

BDG 7.14.01

Check if $l_{dy.avail} > l_{dy.req}$

$$Check := if\left(l_{dy\_avail} > l_{dy.req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Transverse Direction of the Footing

Available length for rebar development

$$l_{dx\_avail} := \frac{w_{footing} - t_{wall}}{2} - Cover_{ft} = 38 \cdot in$$

Assuming that the bars are at high stress, the required development length for No. 6 bars at 12 in. spacing

$$l_{dx.req} := 15in$$

BDG 7.14.01

Check if $l_{dx.avail} > l_{dx.req}$

$$Check := if\left(l_{dx\_avail} > l_{dx.req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Shrinkage and Temperature Reinforcement

This shrinkage and temperature reinforcement requirement for the steel at the bottom of the footing was already checked and the requirements were satisfied.

The reinforcement at the top of the footing should satisfy the shrinkage and temperature reinforcement requirements. **LRFD 5.10.6**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. in base walls and pier footings adjacent to roadways. **BDG 5.22.01**

Select a trial bar size $bar := 5$

Cross-section area of a reinforcing steel bar $A_{bar} := Area(bar) = 0.31 \cdot in^2$

Select a spacing for reinforcing steel bars $s_{barST} := 12 \cdot in$

Provided horizontal reinforcement area $A_{sProvidedST} := \dfrac{A_{bar} \cdot 12in}{s_{barST}} = 0.31 \cdot in^2$

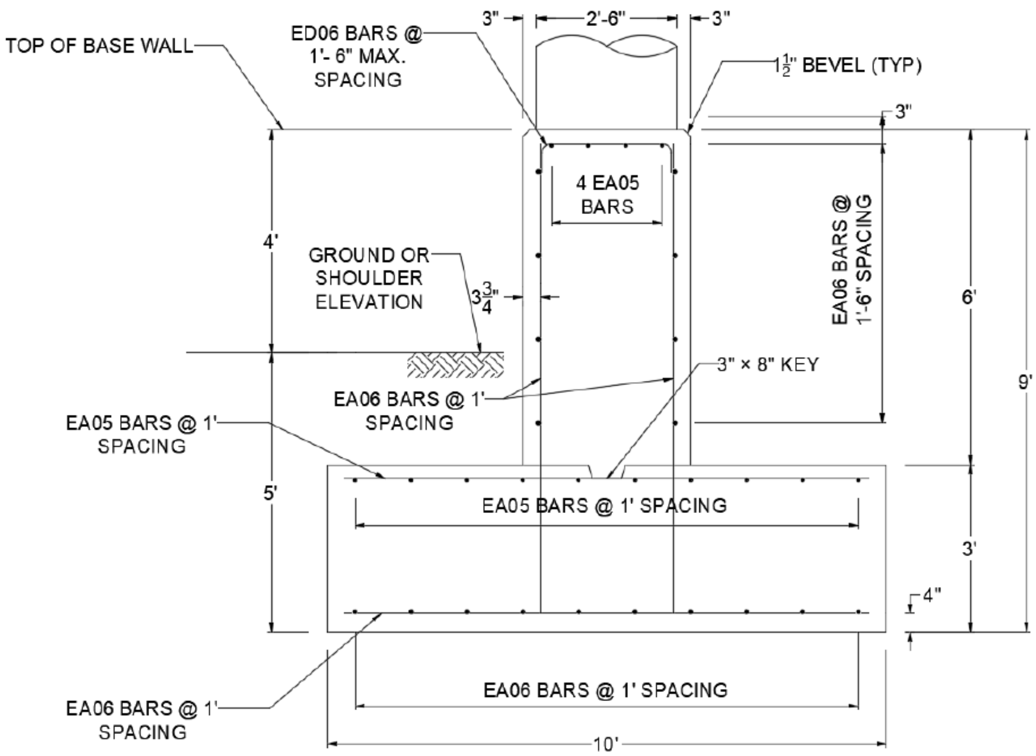
Required shrinkage and temperature steel area in the transverse and longitudinal directions (calculated previously in the flexural design) $A_{shrink.temp} = 0.3 \cdot in^2$

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

Therefore, use No. 5 bars at 12.0 in. spacing ($A_s = 0.31$ in.^{2}/ft) as the shrinkage and temperature reinforcement at the top of the footing in both longitudinal and transverse directions.

The footing and base wall designs resulted in the following details:

- No. 6 bars @ 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the transverse flexural reinforcement at the bottom of the footing
- No. 6 bars @ 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) as the longitudinal flexural reinforcement at the bottom of the footing
- No. 5 bars @ 12.0 in. spacing ($A_s = 0.31$ in.^{2}/ft) as the shrinkage and temperature reinforcement at the top of the footing in both longitudinal and transverse directions
- Four No. 5 bars ($A_s = 1.24$ in.^{2}) at the top of the base wall
- No. 6 bars @ 12.0 in. spacing ($A_s = 0.44$ in.^{2}/ft) on each face of the base wall as the vertical flexural reinforcement
- No. 6 bars @ 18.0 in. spacing ($A_s = 0.293$ in.^{2}/ft) on each face of the base wall as the horizontal shrinkage and temperature reinforcement.



Note: Certain details are not shown in this drawing for clarity of main reinforcement. Refer to MDOT Bridge Design Guides for additional details.

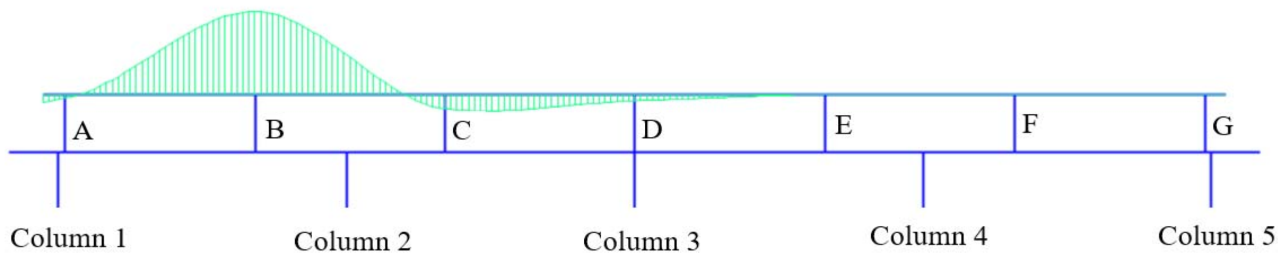
Appendix 5.A Influence Lines

Description

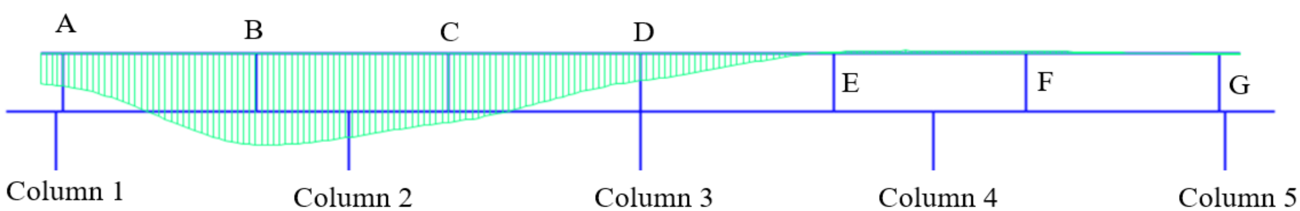
This appendix presents the influence lines used to define the live load positions to develop the maximum load effects in the pier cap.

Structural analysis software is used to generate the following influence lines.

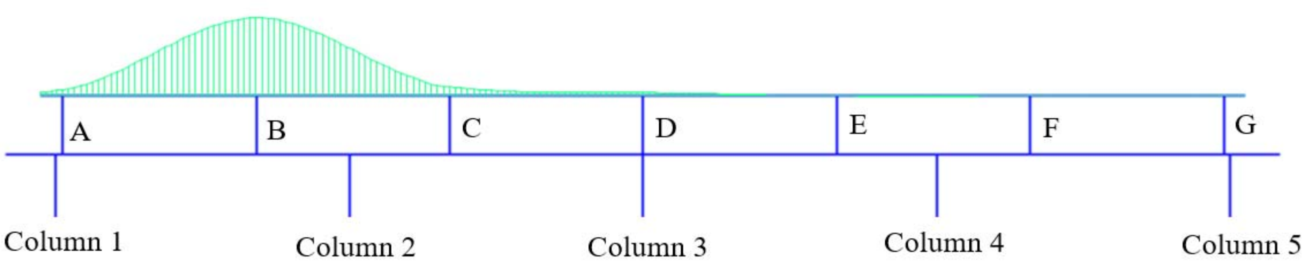
Influence line to develop the maximum Girder B reaction to obtain the maximum positive moment in the first bay of the pier cap



Influence line to develop the maximum girder reactions to obtain the maximum negative moment in the pier cap (the maximum negative moment developed at Column 2)



Influence line to develop the maximum girder reactions to obtain the maximum shear in the pier cap (the maximum shear developed at Column 2)



Appendix 5.B Ice Load

Description

This appendix presents the calculation of ice load on the pier. Ice load only applies to the piers with pile foundations that are located over water ways. The calculation of ice load presented in this appendix is for illustrative purposes only.

The pier in this design example is not subjected to the ice load since the feature intersect is a roadway. However, for illustrative purposes, it is assumed that the bridge crosses a water body and the level of ice action is at the base wall.

All piers subjected to the dynamic or static force of ice shall be designed according to the current AASHTO LRFD Specifications. **BDM 7.01.04G**

The calculation of ice load, IC, on a pier is specified in AASHTO LRFD Article 3.9.

Effective ice crushing strength $p_{ice} := 32\text{ksf}$

Note: The MDOT practice is to use the largest ice crushing strength of 32 ksf, specified in the LRFD. **LRFD C3.9**

The following empirical equation is used to calculate the thickness of ice when the observations over a long period of time are not available.

$$t_{ice} = 0.083 \cdot \alpha \cdot \sqrt{S_f}$$ **LRFD Eq. C3.9.2.2-1**

α - the coefficient for local conditions (taken as 0.5 for an average river with snow) **LRFD C3.9.2.2**

S_f - the freezing index (the algebraic sum, Σ(32-T), from the date of freeze up to the date of interest, in degree days)

T - the mean daily air temperature (degrees F).

Since there is no detailed information available, an assumed ice thickness is used.

Assumed thickness of ice $t_{ice} := 1.5\text{ft}$

Width of the structure at the level of ice action $w := t_{wall} = 3\text{ ft}$

Coefficient accounting for the effect of the structure width/ice thickness ratio where the floe fails by crushing $C_a := \sqrt{\frac{5 \cdot t_{ice}}{w} + 1} = 1.871$ **LRFD Eq. 3.9.2.2-3**

Ice crushing force $F_c := C_a \cdot p_{ice} \cdot t_{ice} \cdot w = 269.399 \cdot \text{kip}$ **LRFD Eq. 3.9.2.2-1**

Inclination angle of pier nose to the vertical (deg.) $\alpha := 37$ **BDG 5.21.01**

Coefficient accounting for the inclination of pier nose with respect to a vertical $C_n := \frac{0.5}{\tan[(\alpha - 15)\text{deg}]} = 1.238$ **LRFD Eq. 3.9.2.2-4**

Ice flexing force $F_b := C_n \cdot p_{ice} \cdot t_{ice}^2 = 89.103 \cdot \text{kip}$ **LRFD Eq. 3.9.2.2-2**

The horizontal ice force $F_{ice} := \text{if}\left(\frac{w}{t_{ice}} > 6, F_c, \min(F_c, F_b)\right) = 89.103 \cdot \text{kip}$ **LRFD Eq. 3.9.2.2**

This force acts along the longitudinal direction of the pier. In the transverse direction of the pier, an ice force of 15% of the longitudinal ice force is used.

$$F_{ice\_T} := 0.15 \cdot F_{ice} = 13.365 \cdot \text{kip}$$

An Extreme Event II load combination should be used to calculate the load effects when considering ice load.

Extreme II = 1.0DC + 1.0DW + 0.5LL + 0.5BR + 1.0IC

Appendix 5.C Centrifugal Force

Description

This appendix presents the calculation of centrifugal force applied on the pier.

Centrifugal force, CE, is specified in LRFD 3.6.3 and is included in the pier design for structures on horizontal curves. The lane load portion of the HL-93 loading is neglected in the computation of the centrifugal force.

Although the bridge in this example is not a curved bridge, for illustrative purposes, a curved bridge is considered.

The centrifugal effect on live load shall be taken as the product of the axle weights of the design truck or tandem and a factor C.

Assumed highway design speed $v := 70\,\frac{mile}{hr} = 102.667\,\frac{ft}{s}$

Gravitational acceleration $g = 32.174\,\frac{ft}{s^2}$

Assumed radius of curvature of traffic lane $R := 3500ft$

Centrifugal load factor $C := \frac{4 \cdot v^2}{3g \cdot R} = 0.125$ **LRFD Eq. 3.6.3.1**

The design truck, HL-93, has a total axle weight of 72 kips. The Michigan modification factor of 1.2 for HL-93MOD (i.e., $f_{HL93Mod}$) should be considered.

Centrifugal force $CE := C \cdot 72kip \cdot f_{HL93Mod} = 10.783 \cdot kip$

This centrifugal force shall be applied radially and horizontally at a distance 6 ft above the roadway surface.

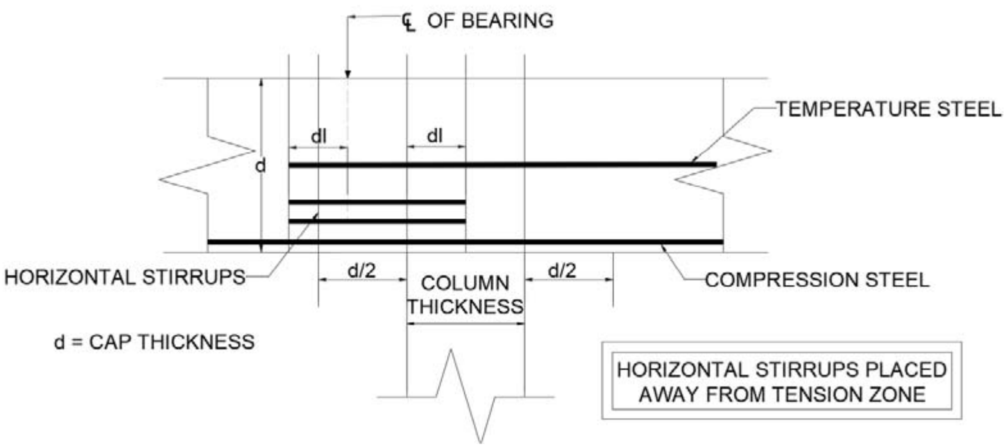
Appendix 5.D Shear Friction Reinforcement

Description

This appendix presents the design of shear friction reinforcement in the pier cap.

MDOT practice requires shear-friction reinforcement when a bearing is located within a distance d/2 from the face of the column, where d is the depth of pier cap.

The horizontal stirrup (shear-friction reinforcement) steel area is equal to the area required by the shear at the face of the column minus the skin reinforcement in the cap.



In this example, none of the bearing points are located within a distance of d/2 from the face of the pier columns. However, for illustrative purposes, it is assumed that a bearing point lies within a distance of d/2 from the column face.

Maximum factored shear force at the face of Column 2 (from structural analysis) $V_{uPCapStrI} = 475.5 \cdot kip$

For normal weight concrete placed monolithically with the column **LRFD 5.7.4.4**

Cohesion factor $c := 0.4ksi$

Friction factor $\mu := 1.4$

Fraction of concrete strength available to resist interface shear $K_1 := 0.25$

Limiting interface shear resistance $K_2 := 1.5ksi$

Interface area between the pier cap and the column $A_{cv} := h_{cap} \cdot t_{cap} = 9.625\ ft^2$

As per the details in Step 5.6, 5 No. 4 bars evenly distributed on each face of the pier cap are used as the skin reinforcement.

Next, check if the skin reinforcement is sufficient as shear friction steel. If not, additional shear friction steel needs to be added.

$bar := 4$ $A_{bar} := Area(bar) = 0.2 \cdot in^2$

Number of bars provided as the skin reinforcement $N_{bsk} := 10$

Area of skin reinforcement $A_{vf1} := N_{bsk} \cdot A_{bar} = 2 \cdot in^2$

The compression steel at the bottom of the cap has 6 No. 9 bars.

$bar := 9$ $A_{bar} := Area(bar) = 1 \cdot in^2$

Number of bars provided on the compression side as the skin reinforcement

$N_{bc.sk} := 6$

Area of skin reinforcement

$A_{vf2} := N_{bc.sk} \cdot A_{bar} = 6 \cdot in^2$

The total area of interface shear reinforcement crossing the shear plane

$$A_{vf} := A_{vf1} + A_{vf2} = 8 \cdot in^2$$

Permanent net compressive force normal to the shear plane can be assumed to be zero.

$$P_c := 0$$

The nominal shear resistance of the interface plane

$$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 1.226 \times 10^3 \cdot kip$$

LRFD Eq. 5.7.4.3-3

The nominal shear resistance, V_{ni}, shall not exceed either of the following:

$$V_{ni1} := K_1 \cdot f_c \cdot A_{cv} = 1.04 \times 10^3 \cdot kip$$

LRFD Eq. 5.7.4.3-4

$$V_{ni2} := K_2 \cdot A_{cv} = 2.079 \times 10^3 \cdot kip$$

LRFD Eq. 5.7.4.3-5

$$V_{niControl} := \min\left(V_{ni}, V_{ni1}, V_{ni2}\right) = 1.04 \times 10^3 \cdot kip$$

Resistance factor for shear

$\phi_v = 0.9$

LRFD 5.5.4.2

Factored shear resistance

$V_{ri} := \phi_v \cdot V_{niControl} = 935.55 \cdot kip$

Check if the factored shear resistance > the shear demand

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

Therefore, no additional shear friction steel is required.

Section 6 Multi-Column Pier with Pile Foundation

Step 6.1 Preliminary Dimensions

Description

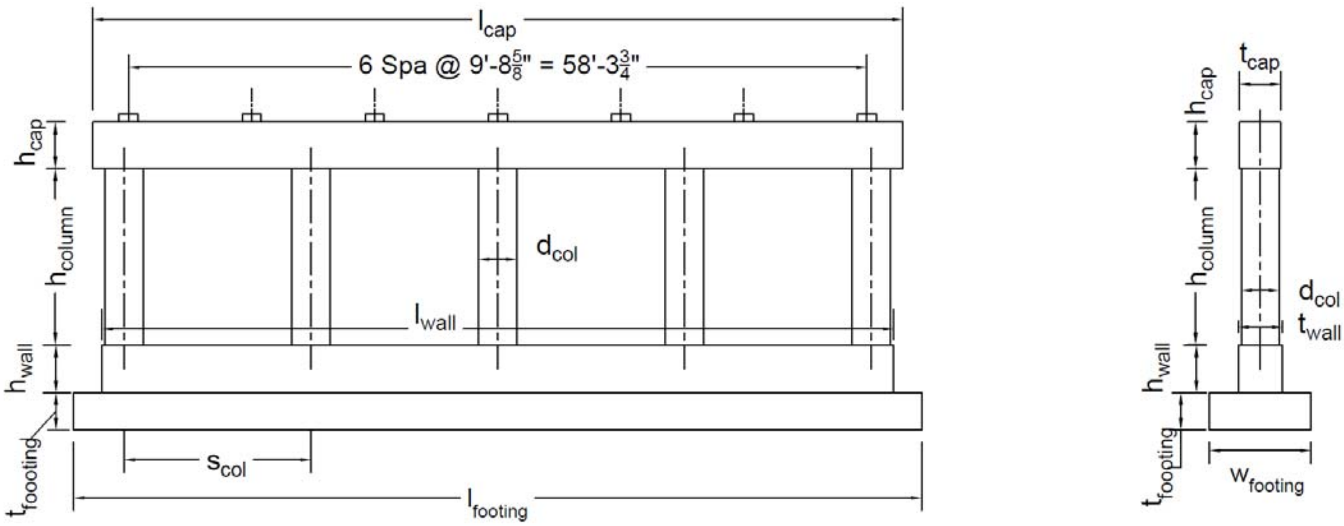
This step presents the selected preliminary dimensions.

Refer to Section 2 of the *Design of Highway Bridge Abutments and Foundations Example* developed by Attanayake and Hu (2023) for the design criteria, bridge information, material properties, and soil types and properties.

The preliminary dimensions are selected based on site-specific conditions, highway agency standards, and past experience.

Construction joints should be provided when the pier cap is longer than 25 ft. **BDM 7.03.03.C.3**
A 1-in. open joint may be required to control temperature moment in long piers with short columns.

The following figure shows the pier geometry and its associated dimensional variables:



The preliminary dimensions selected for this example are given below.

Pier cap length $l_{cap} := 64ft$

Pier cap height $h_{cap} := 3.5ft$

The pier cap is to be approximately 3 in. wider than the column diameter and should provide a 4.5 in. minimum clearance between the edge of masonry plate (or elastomeric pad) and the face of the cap. **BDM 7.03.03.C.1**

Pier cap thickness $t_{cap} := 2.75ft$

Pier cap overhang length $Overhang_{cap} := 2.5ft$

Column spacing $S_{col} := 14.75ft$ **BDM 7.03.03.B.4**

In general, 3 ft diameter columns should be used with 42 in. or greater beam depths and 2.5 ft diameter columns with beams less than 42 in., unless loading conditions or bearing areas dictate larger columns. **BDM 7.03.03.B.1**

Beam depth $b_{depth} := 33in$

Column diameter $d_{col} := 2.5ft$

Clear height of the column $h_{col} := 12ft$

No. of columns $n_{col} := 5$

Piers that are within the clear zone or in a median where barriers are required should have base walls. The base wall is to be 3 in. wider than the column to prevent vehicle snagging and should extend a minimum of 3.5 ft above the ground. **BDM 7.03.03.D**

BDG 5.22.01 and 5.24.01 provide base wall details for piers located adjacent to roadways and railway tracks, respectively. **BDG 5.22.01** **BDG 5.24.01**

According to BDG 5.22.01, the base wall should be 6 in. wider than the column; this contradicts with the BDM 7.03.03.D requirements. This example uses a base wall that is 6 in. wider than the column diameter.

For piers located adjacent to roadways, the minimum height of the base wall shall be 5 ft with a minimum of 3.5 ft extended above the ground or shoulder. For piers located adjacent to railway tracks, the minimum height of the base wall shall be 11ft with a minimum of 6 ft extended above the top of the rail head. Designers shall refer to BDG 5.24 for additional requirements for bridge piers located near the railway tracks.

This example demonstrates the design of a pier located adjacent to a roadway and supported on pile foundation.

The difference between base wall thickness and column diameter	$diff_{wall} := 6in$
Base wall thickness	$t_{wall} := d_{col} + diff_{wall} = 3\,ft$
Base wall length	$l_{wall} := (n_{col} - 1)\cdot S_{col} + d_{col} + 6in = 62\,ft$
Base wall height above the footing top	$h_{wall} := 6ft$
Footing length	$l_{footing} := 67ft$

The minimum footing thickness for bridge piers located adjacent to roadways and railway tracks is 2 ft 6 in. **BDG 5.22.01** **BDG 5.24.01**

Footing thickness	$t_{footing} := 3ft$
Footing width	$w_{footing} := 8ft$
Depth of soil above the footing top	$h_{soil} := 2ft$

Note: The depth from the ground level to the bottom of the footing needs to be maintained at a minimum of 4 ft. for frost depth. Typically, a 1-ft deep soil profile is maintained with normal grading when the pier is at a median. The depth of the soil may change to 2 to 3 ft based on the pavement profile when the pier is closer to the pavement.

Concrete Cover Requirements for Reinforcing Steel

Unless otherwise shown on the plans, the minimum concrete clear cover for reinforcement shall satisfy the following requirements: **BDM 8.02.N**

For concrete cast against earth: 3 in.
For all other cases unless shown on plans: 2 in.

The following concrete cover dimensions are selected using BDG 5.22.01 as examples: **BDG 5.22.01**

Cover for the base wall side	$Cover_{wall} := 3.75in$
Cover for the base wall top	$Cover_{walltop} := 3in$
Cover for the top and side of footing	$Cover_{ft} := 4in$

Since the concrete cover requirements for pier caps and columns are not provided in the BDM and BDG, the following dimensions are taken from the MDOT Sample Bridge Plans.

Cover for the bend cap $Cover_{cap} := 3.5in$

Cover for the columns $Cover_{col} := 4in$

Step 6.2 Application of Dead Load

Description

This step describes the application of dead loads on the pier.

Dead Load Girder Reactions

The superstructure dead load reactions per bearing are taken from the *Steel Plate Girder Design Example*. All the beam seats are assumed to be at the same elevation.

When calculating superstructure loads on the substructure, 75% of the barrier dead load should be applied with the fascia beam load. The remaining 25% of the barrier load should be applied with the first interior girder load. **BDM 7.01.04.J**

Note: The exterior and interior girder shear values presented in the *Steel Plate Girder Design Example* (Table 12 and 13) were calculated by equally distributing the barrier loads to all the girders. Therefore, the girder reactions over the pier due to barrier loads need to be recalculated as shown below.

<u>Exterior Girders</u> **Table 12 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DCEx\_noBarrier} := 161.4kip$$

Reaction due to 75% of the barrier weight (DB) on the exterior girder $R_{DCEx\_barrier} := 44kip$

Total exterior girder reaction due to DC $R_{DCEx} := R_{DCEx\_noBarrier} + R_{DCEx\_barrier} = 205.4 \cdot kip$

Reaction due to the weight of the future wearing surface (DW) $R_{DWEx} := 26.6kip$

<u>First Interior Girder</u> **Table 13 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DC1stIn\_noBarrier} := 190.4kip$$

Reaction due to 25% of the barrier weight (DB) on the first interior girder $R_{DC1stIn\_barrier} := 14.5kip$

Total first interior girder reaction due to DC $R_{DC1stInt} := R_{DC1stIn\_noBarrier} + R_{DC1stIn\_barrier} = 204.9 \cdot kip$

Reaction due to the future wearing surface weight (DW) $R_{DWIn} := 26.4kip$

<u>Other Interior Girders</u> **Table 13 of the Steel Plate Girder Design Example**

Reaction due to the weight of structural components and non-structural attachments (DC), including the stay-in-place formwork but excluding barrier weight

$$R_{DCIn} := 190.4kip$$

Dead Load Calculation

Dead load of superstructure

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

$$DC_{Sup} := 2 \cdot R_{DCEx} + 2 \cdot R_{DC1stInt} + \left(N_{beams} - 4\right) \cdot R_{DCIn}$$

$$DC_{Sup} = 1.392 \times 10^{3} \cdot kip$$

Weight of the future wearing surface (DW)

$$DW_{Sup} := 2 \cdot R_{DWEx} + \left(N_{beams} - 2\right) \cdot R_{DWIn} = 185.2 \cdot kip$$

Pier cap weight $DC_{cap} := l_{cap} \cdot h_{cap} \cdot t_{cap} \cdot W_c = 92.4 \cdot kip$

Pier column weight $DC_{column} := n_{col} \frac{1}{4} \pi \, d_{col}^{\;2} \cdot h_{col} \cdot W_c = 44.179 \cdot kip$

Base wall weight $DC_{wall} := l_{wall} \cdot t_{wall} \cdot h_{wall} \cdot W_c = 167.4 \cdot kip$

Footing weight $DC_{footing} := w_{footing} \cdot l_{footing} \cdot t_{footing} \cdot W_c = 241.2 \cdot kip$

Step 6.3 Application of Live Load

Description

The live load application procedure and relevant calculations are described in Step 5.3.

Step 6.4 Application of Other Loads

Description

The application of other loads include braking force, wind load, temperature load, earth load, and vehicle collision load is discussed in Step 5.4. Ice load and centrifugal force are not applicable for this example. For illustrative purposes, the calculation of ice load and centrifugal force is given in Appendix 5.B and 5.C.

Step 6.5 Combined Load Effects

Description

This step presents the procedure of combining all load effects and calculates the total factored forces and moments acting on the pier cap, columns, base wall, and footing.

Since the combined loadings on the pier cap, columns, and base wall are identical to Step 5.5, only the calculation of combined load effects at the base of the footing is presented.

Strength I, Strength III, Strength V, and Service I limit states are considered for the analysis and design of the pier. **LRFD 3.4.1**

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

Strength III = 1.25DC + 1.5DW + 1.5EH + 1.35EV + 1.0WS + 0.5TU

Strength V = 1.25DC + 1.5DW + 1.35LL + 1.35BR + 1.0WS + 1.0WL + 1.5EH + 1.35EV + 1.35LS + 0.5TU

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

- BR = vehicular braking force
- DC = dead load of structural components and nonstructural attachments
- DW = dead load of future wearing surface and utilities
- EH = horizontal earth pressure load
- EV = vertical pressure from the earth fill
- LL = vehicular live load
- LS = live load surcharge
- WL = wind on live load
- WS = wind load on structure
- TU = force effect due to uniform temperature

Limit states that are not shown either do not control or are not applicable.

Note: These load combinations should include the maximum and minimum load factors; only the maximum factors are shown for clarity.

Forces and Moments at the Pier Footing

The bearing pressure distribution depends on the rigidity of the footing and the soil type and condition. The pier footings are usually rigid, and the assumption $q = (P/A) +/- (Mc/I)$ is valid. For an accurate calculation of bearing pressure distribution, the footing may be analyzed as a beam on an elastic foundation.

The braking force, wind load on the superstructure, and wind load acting on the live load are applied at the bearings.

Moment arm from the top of the pier cap to the bottom of the base wall $\quad Arm_{wall} := h_{cap} + h_{col} + h_{wall} = 21.5\ ft$

The live load on all five lanes develops the critical load effects for the footing design.

Moment arm of Girder A and G reactions to the center of footing $\quad Arm_{AG} := 3S = 29.156\ ft$

Moment arm of Girder B and F reactions to the center of footing $\quad Arm_{BF} := 2S = 19.438\ ft$

Moment arm of Girder C and E reactions to the center of footing $\quad Arm_{CE} := S = 9.719\ ft$

Strength I

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

Factored vertical force

$$F_{VFtStrI} := 1.25\cdot\left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \dots + 1.5DW_{Sup} + 1.75R_{LLFooting} + 1.35\cdot EV_{Ft}$$

$$F_{VFtStrI} = 3.55\times 10^{3}\cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrI} := 0$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrI} := 1.75\cdot BRK_{5L} = 56.875\cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtStrI} := 1.75\cdot BRK_{5L}\cdot\left(Arm_{wall} + t_{footing}\right)$$

$$M_{XFtStrI} = 1.393\times 10^{3}\cdot kip\cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtStrI} := 1.75\cdot\left[\left(R_{GFt\_5L} - R_{AFt\_5L}\right)\cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right)\cdot Arm_{BF} + \left(R_{EFt\_5L} - R_{CFt\_5L}\right)\cdot Arm_{CE}\right]$$

$$M_{YFtStrI} = 894.546\cdot kip\cdot ft$$

Strength III

Strength III = 1.25DC + 1.5DW + 1.5EH + 1.35EV + 1.0WS + 0.5TU

Factored vertical force

$$F_{VFtStrIII} := 1.25\cdot\left(DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}\right) \dots + 1.5DW_{Sup} + 1.35\cdot EV_{Ft}$$

$$F_{VFtStrIII} = 2.812\times 10^{3}\cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrIII} := N_{beams}\cdot WS_{TStrIII} + P_{TPierCap.StrIII} + W_{TCol.StrIII}\cdot h_{col} + P_{TWall.StrIII} = 24.087\cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrIII} := N_{beams}\cdot WS_{LStrIII} + W_{LPierCap.StrIII}\cdot l_{cap} + n_{col}\cdot W_{LCol.StrIII}\cdot h_{col} + P_{LWall.StrIII}$$

$$V_{LFtStrIII} = 34.941\cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtStrIII} := N_{beams}\cdot WS_{LStrIII}\cdot\left(Arm_{wall} + t_{footing}\right) \dots + W_{LPierCap.StrIII}\cdot l_{cap}\cdot\left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \dots + n_{col}\cdot W_{LCol.StrIII}\cdot h_{col}\cdot\left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.StrIII}\cdot\left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{XFtStrIII} = 619.589\cdot kip\cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtStrIII} := N_{beams} \cdot WS_{TStrIII} \cdot (Arm_{wall} + t_{footing}) \dots \\ + P_{TPierCap.StrIII} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \dots \\ + W_{TCol.StrIII} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.StrIII} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{YFtStrIII} = 570.47 \cdot kip \cdot ft$$

Strength V

Strength V = 1.25DC + 1.5DW + 1.35LL + 1.35BR + 1.0WS + 1.0WL + 1.5EH + 1.35EV + 1.35 LS + 0.5TU

Factored vertical force

$$F_{VFtStrV} := 1.25 \cdot (DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing}) \dots \\ + 1.5DW_{Sup} + 1.35 \cdot R_{LLFooting} + 1.35 \cdot EV_{Ft}$$

$$F_{VFtStrV} = 3.382 \times 10^{3} \cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrV} := N_{beams} \cdot (WS_{TStrV} + WL_{TBearing}) + P_{TPierCap.StrV} \dots \\ + W_{TCol.StrV} \cdot h_{col} + P_{TWall.StrV}$$

$$V_{TFtStrV} = 26.44 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrV} := 1.35 \cdot BRK_{5L} + N_{beams} \cdot (WS_{LStrV} + WL_{LBearing}) \dots \\ + W_{LPierCap.StrV} \cdot l_{cap} + n_{col} \cdot W_{LCol.StrV} \cdot h_{col} + P_{LWall.StrV}$$

$$V_{LFtStrV} = 75.724 \cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtStrV} := 1.35 \cdot BRK_{5L} \cdot (Arm_{wall} + t_{footing}) \dots \\ + N_{beams} \cdot \left[WS_{LStrV} \cdot (Arm_{wall} + t_{footing}) + WL_{LBearing} \cdot (Arm_{wall} + t_{footing})\right] \dots \\ + W_{LPierCap.StrV} \cdot l_{cap} \cdot \left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \dots \\ + n_{col} \cdot W_{LCol.StrV} \cdot h_{col} \cdot \left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.StrV} \cdot \left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{XFtStrV} = 1.694 \times 10^{3} \cdot kip \cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtStrV} := 1.35\left[\left(R_{GFt\_5L} - R_{AFt\_5L}\right)\cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right)\cdot Arm_{BF} \cdots + \left(R_{EFt\_5L} - R_{CFt\_5L}\right)\cdot Arm_{CE}\right] \cdots + N_{beams}\cdot\left[WS_{TStrV}\cdot\left(Arm_{wall} + t_{footing}\right) + WL_{TBearing}\cdot\left(Arm_{wall} + t_{footing}\right)\right] \cdots + P_{TPierCap.StrV}\cdot\left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots + W_{TCol.StrV}\cdot h_{col}\cdot\left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.StrV}\cdot\left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{YFtStrV} = 1.324 \times 10^{3}\cdot kip\cdot ft$$

Service I

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

Factored vertical force

$$F_{VFtSerI} := DC_{Sup} + DC_{cap} + DC_{column} + DC_{wall} + DC_{footing} \cdots + DW_{Sup} + R_{LLFooting} + EV_{Ft}$$

$$F_{VFtSerI} = 2.628 \times 10^{3}\cdot kip$$

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtSerI} := N_{beams}\cdot\left(WS_{TSerI} + WL_{TBearing}\right) + P_{TPierCap.SerI} \cdots + W_{TCol.SerI}\cdot h_{col} + P_{TWall.SerI}$$

$$V_{TFtSerI} = 22.587\cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtSerI} := BRK_{5L} + N_{beams}\cdot\left(WS_{LSerI} + WL_{LBearing}\right) \cdots + W_{LPierCap.SerI}\cdot l_{cap} + n_{col}\cdot W_{LCol.SerI}\cdot h_{col} + P_{LWall.SerI}$$

$$V_{LFtSerI} = 58.759\cdot kip$$

Factored moment about the longitudinal axis of the footing

$$M_{XFtSerI} := BRK_{5L}\cdot\left(Arm_{wall} + t_{footing}\right) \cdots + N_{beams}\cdot\left[WS_{LSerI}\cdot\left(Arm_{wall} + t_{footing}\right) + WL_{LBearing}\cdot\left(Arm_{wall} + t_{footing}\right)\right] \cdots + W_{LPierCap.SerI}\cdot l_{cap}\cdot\left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots + n_{col}\cdot W_{LCol.SerI}\cdot h_{col}\cdot\left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{LWall.SerI}\cdot\left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{XFtSerI} = 1.316 \times 10^{3}\cdot kip\cdot ft$$

Factored moment about the transverse axis of the footing

$$M_{YFtSerI} := \left(R_{GFt\_5L} - R_{AFt\_5L}\right)\cdot Arm_{AG} + \left(R_{FFt\_5L} - R_{BFt\_5L}\right)\cdot Arm_{BF} \cdots$$
$$+ \left(R_{EFt\_5L} - R_{CFt\_5L}\right)\cdot Arm_{CE} \cdots$$
$$+ N_{beams}\cdot\left[WS_{TStrV}\cdot\left(Arm_{wall} + t_{footing}\right) + WL_{TBearing}\cdot\left(Arm_{wall} + t_{footing}\right)\right] \cdots$$
$$+ P_{TPierCap.SerI}\cdot\left(\frac{h_{cap}}{2} + h_{col} + h_{wall} + t_{footing}\right) \cdots$$
$$+ W_{TCol.SerI}\cdot h_{col}\cdot\left(\frac{h_{col}}{2} + h_{wall} + t_{footing}\right) + P_{TWall.SerI}\cdot\left(\frac{h_{wall} + h_{soil}}{2} + t_{footing}\right)$$

$$M_{YFtSerI} = 1.141 \times 10^{3}\cdot kip\cdot ft$$

The designs of the pier cap, columns, and the base wall are presented in Steps 5.6, 5.7, and 5.8. The subsequent steps of this example present the design of piles and the footing.

Step 6.6 Pile Design

Description

This step presents the selection of pile type, the design of pile size and layout, and the evaluation of pile's lateral force resistance.

Page Content

Pile Size and Layout Design

This example uses steel H piles since they are 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 $\quad Pile\_embd := 6in \quad$ **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**
2. Edge distance: The typical minimum edge distance for piles is 18 inches. **BDM 7.03.09.A7**

Pile edge distance $\quad PileEdgeDist := 18in$

Number of rows of piles $\quad Pile_{row} := 2$

Number of piles in each row $\quad PilesInEachRow := 9$

Total number of piles $\quad N_{piles} := Pile_{row} \cdot PilesInEachRow = 18$

Pile spacing in the direction parallel to x-axis

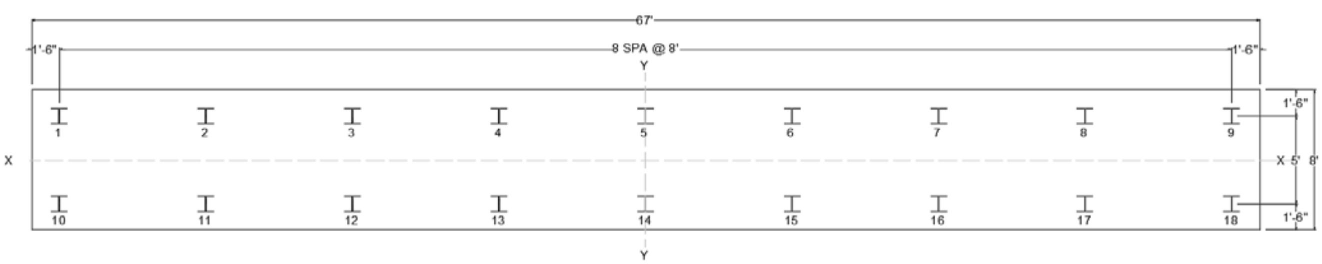
$$Spacing_x := \frac{l_{footing} - 2PileEdgeDist}{PilesInEachRow - 1} = 8 \cdot ft$$

Pile spacing in the direction parallel to y-axis

$$Spacing_y := \frac{w_{footing} - 2PileEdgeDist}{Pile_{row} - 1} = 5\,ft$$

Since the depth of commonly used H-pile cross-sections ranges from 10 to 14 inches, the above pile spacings satisfy the minimum pile spacing requirement of 30 to 42 inches (i.e. 2.5 ft to 3.5 ft).

The preliminary pile layout is shown below.



Section modulus of the pile group about x-axis

$$S_{XX} := N_{piles} \cdot \frac{(0.5Spacing_y)^2}{0.5Spacing_y} = 45 \cdot ft$$

Section modulus of the pile group about y-axis

$$S_{YY} := 4 \cdot \frac{(Spacing_x)^2 + (2 \cdot Spacing_x)^2 + (3 \cdot Spacing_x)^2 + (4 \cdot Spacing_x)^2}{4 \cdot Spacing_x} = 240\,ft$$

Strength I

$F_{VFtStrI} = 3.55 \times 10^3 \cdot kip$ $M_{XFtStrI} = 1.393 \times 10^3 \cdot kip \cdot ft$ $M_{YFtStrI} = 894.546 \cdot kip \cdot ft$

Maximum pile reaction $P_{uMax\_StrI} := \dfrac{F_{VFtStrI}}{N_{piles}} + \dfrac{M_{XFtStrI}}{S_{XX}} + \dfrac{M_{YFtStrI}}{S_{YY}} = 231.926 \cdot kip$

Strength III

$F_{VFtStrIII} = 2.812 \times 10^3 \cdot kip$ $M_{XFtStrIII} = 619.589 \cdot kip \cdot ft$ $M_{YFtStrIII} = 570.47 \cdot kip \cdot ft$

Maximum pile reaction $P_{uMax\_StrIII} := \dfrac{F_{VFtStrIII}}{N_{piles}} + \dfrac{M_{XFtStrIII}}{S_{XX}} + \dfrac{M_{YFtStrIII}}{S_{YY}} = 172.391 \cdot kip$

Strength V

$F_{VFtStrV} = 3.382 \times 10^3 \cdot kip$ $M_{XFtStrV} = 1.694 \times 10^3 \cdot kip \cdot ft$ $M_{YFtStrV} = 1.324 \times 10^3 \cdot kip \cdot ft$

Maximum pile reaction $P_{uMax\_StrV} := \dfrac{F_{VFtStrV}}{N_{piles}} + \dfrac{M_{XFtStrV}}{S_{XX}} + \dfrac{M_{YFtStrV}}{S_{YY}} = 231.024 \cdot kip$

Service I

Pile reactions under the Service I limit state are needed in the flexural design of the footing.

$F_{VFtSerI} = 2.628 \times 10^3 \cdot kip$ $M_{XFtSerI} = 1.316 \times 10^3 \cdot kip \cdot ft$ $M_{YFtSerI} = 1.141 \times 10^3 \cdot kip \cdot ft$

Maximum pile reaction $P_{uMax\_SerI} := \dfrac{F_{VFtSerI}}{N_{piles}} + \dfrac{M_{XFtSerI}}{S_{XX}} + \dfrac{M_{YFtSerI}}{S_{YY}} = 179.986 \cdot kip$

The controlling maximum pile reaction $P_{uMax} := \max\left(P_{uMax\_StrI}, P_{uMax\_StrIII}, P_{uMax\_StrV}\right) = 231.926 \cdot kip$

Nominal pile resistance of commonly used steel H-piles **BDM 7.03.09.B.1**

HP 10X42	275 kips
HP 10X57	350 kips
HP 12X53	350 kips
HP 12X74	500 kips
HP 12X84	600 kips
HP 14X73	500 kips
HP 14X89	600 kips

The resistance factor for driven piles assuming that the nominal pile driving resistance is verified using the FHWA-modified Gates Dynamic Formula **BDM 7.03.09.B2**

$\varphi_{dyn} := 0.5$

Required minimum nominal pile resistance $R_{nReq} := \dfrac{P_{uMax}}{\varphi_{dyn}} = 463.852 \cdot kip$

Selected pile section HP 14X73 $b_f := 14.585in$ $d_{pile} := 13.61in$

Minimum spacing between piles in the selected layout

$PileSpacing_{min} := \min(Spacing_x, Spacing_y) = 60 \cdot in$

Check if the spacing of the piles is greater than $3d_{pile}$

$Check := if(PileSpacing_{min} > 3d_{pile}, "OK", "Not OK") = "OK"$

Consult the Geotechnical Services Section for the nominal resistance of the selected section. **BDM 7.03.09.B**

Nominal pile resistance

$R_n := 500kip$

Factored nominal pile resistance

$R_R := \varphi_{dyn} \cdot R_n = 250 \cdot kip > P_{uMax} = 231.926 \cdot kip$ OK

Lateral Force Resistance of Piles

The lateral forces acting on the pier are assumed to be equally shared by the piles. Step 6.5 presents the lateral force calculations.

Note: Per MDOT practice, the typical lateral force resistance of a vertical pile is 12 kips. A pile bending (p-y) analysis may be performed by incorporating soil-pile interaction to determine a more accurate lateral force resistance. Consult the Geotechnical Services Section for more information.

Lateral force resistance of a pile

$P_{latProvided} := 12kip$

Strength I

Factored shear force parallel to the transverse axis of the bridge

$V_{TFtStrI} = 0$

Factored shear force parallel to the longitudinal axis of the bridge

$V_{LFtStrI} = 56.875 \cdot kip$

Required pile lateral force resistance parallel to the minor axis of the section (demand)

$$P_{ReqLatMinor\_StrI} := \frac{V_{TFtStrI}}{N_{piles}} = 0 \cdot kip$$

Check if the lateral force resistance > the demand

$Check := if(P_{latProvided} > P_{ReqLatMinor\_StrI}, "OK", "Not OK") = "OK"$

Required pile lateral force resistance parallel to the major axis of the section (demand)

$$P_{ReqLatMajor\_StrI} := \frac{V_{LFtStrI}}{N_{piles}} = 3.16 \cdot kip$$

Check if the pile lateral force resistance > the demand

$Check := if(P_{latProvided} > P_{ReqLatMajor\_StrI}, "OK", "Not OK") = "OK"$

Strength III

Factored shear force parallel to the transverse axis of the bridge

$V_{TFtStrIII} = 24.087 \cdot kip$

Factored shear force parallel to the longitudinal axis of the bridge

$V_{LFtStrIII} = 34.941 \cdot kip$

Required pile lateral force resistance parallel to the minor axis of the section (demand)

$$P_{ReqLatMinor\_StrIII} := \frac{V_{TFtStrIII}}{N_{piles}} = 1.338 \cdot kip$$

Check if the lateral force resistance > the demand

$Check := if(P_{latProvided} > P_{ReqLatMinor\_StrIII}, "OK", "Not OK") = "OK"$

Required lateral force resistance parallel to the major axis of the section (demand)

$$P_{ReqLatMajor\_StrIII} := \frac{V_{LFtStrIII}}{N_{piles}} = 1.941 \cdot kip$$

Check if the lateral force resistance > the demand

$$Check := if\left(P_{latProvided} > P_{ReqLatMajor\_StrIII}, "OK", "Not OK"\right) = "OK"$$

Strength V

Factored shear force parallel to the transverse axis of the bridge

$$V_{TFtStrV} = 26.44 \cdot kip$$

Factored shear force parallel to the longitudinal axis of the bridge

$$V_{LFtStrV} = 75.724 \cdot kip$$

Required lateral force resistance parallel to the minor axis of the section (demand)

$$P_{ReqLatMinor\_StrV} := \frac{V_{TFtStrV}}{N_{piles}} = 1.469 \cdot kip$$

Check if the lateral force resistance > the demand

$$Check := if\left(P_{latProvided} > P_{ReqLatMinor\_StrV}, "OK", "Not OK"\right) = "OK"$$

Required lateral force resistance parallel to the major axis of the section (demand)

$$P_{ReqLatMajor\_StrV} := \frac{V_{LFtStrV}}{N_{piles}} = 4.207 \cdot kip$$

Check if the lateral force resistance > the demand

$$Check := if\left(P_{latProvided} > P_{ReqLatMajor\_StrV}, "OK", "Not OK"\right) = "OK"$$

Step 6.7 Structural Design of the Footing

Description

This step presents the structural design of the pier footing.

Page Contents

Design for Flexure

Transverse Reinforcement

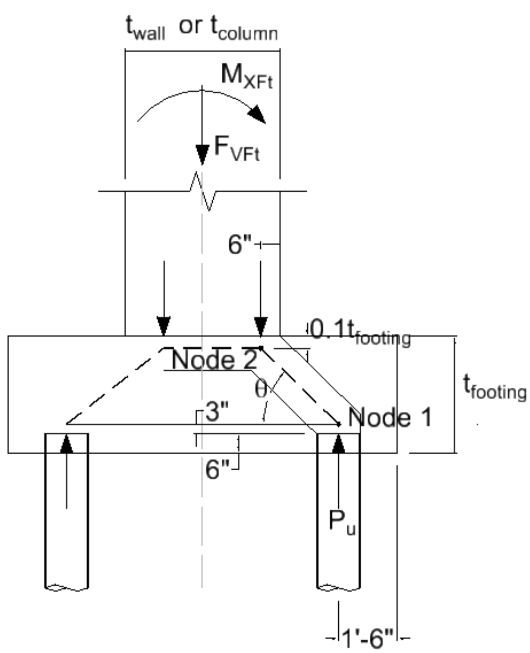
The Strut-and-Tie Method (STM) is used for the design of deep footing and pile caps when the distance between the centers of applied load and the supporting reactions is less than two times the member depth. Therefore, the STM is used in the design of the footing in the transverse direction. **LRFD 5.8.2.1**

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

Distance between the wall or column vertical reaction and a row of piles $S_{center} := \left(\frac{w_{footing}}{2} - PileEdgeDist\right) = 2.5\,ft$

Check if the STM is a suitable model for this footing $Check := if\left(S_{center} < 2t_{footing}, \text{"Use STM"}, \text{"No"}\right) = \text{"Use STM"}$

The following figure shows the STM model selected for the design of the footing in the transverse direction:



The centroid of the top chord is assumed to be located at a distance of 1/10th the footing thickness below the top of the footing. There are several options that the designer may consider when placing the top chord. Please refer to **FHWA-NHI-17-071** *Strut-and-Tie Model (STM) for Concrete Structures* for additional details. Also, Step 7.6 of this series of examples provides more details on this topic.

The tension tie is located at the centroid of the reinforcement that carries the tensile force at the bottom of the footing. The tensile reinforcement is located at 3 in. above the top of the piles. **LRFD C5.8.2.2**

Distance from the top of pile to the center of the transverse reinforcing steel bar $d_R := 3in$

Select a trial bar size $bar := 7$

Nominal diameter of a reinforcing steel bar $d_{bx} := Dia(bar) = 0.875 \cdot in$

Cross-section area of a bar on the flexural tension side $A_{bar} := Area(bar) = 0.6 \cdot in^2$

Projected horizontal length of the strut $l_a := \frac{(w_{footing} - t_{wall})}{2} - PileEdgeDist + 6in = 1.5\ ft$

Projected vertical length of the strut $h_a := t_{footing} - Pile\_embd - 3in - 0.1 \cdot t_{footing} = 1.95\ ft$

Angle between the strut and tension tie $\theta := atan\left(\frac{h_a}{l_a}\right) = 52.431 \cdot °$

Tension Tie Reinforcement Design

As shown below, the first step is to calculate the average pile reaction in a row under strength and service limit states.

Average reaction of a pile in a row, Strength I $P_{RowAvg\_StrI} := \frac{F_{VFtStrI}}{N_{piles}} + \frac{M_{XFtStrI}}{S_{XX}} = 228.199 \cdot kip$

Average reaction of a pile in a row, Strength III $P_{RowAvg\_StrIII} := \frac{F_{VFtStrIII}}{N_{piles}} + \frac{M_{XFtStrIII}}{S_{XX}} = 170.014 \cdot kip$

Average reaction of a pile in a row, Strength V $P_{RowAvg\_StrV} := \frac{F_{VFtStrV}}{N_{piles}} + \frac{M_{XFtStrV}}{S_{XX}} = 225.506 \cdot kip$

Controlling average reaction of the piles in a row under strength limit states

$$P_{RowAvg\_Str} := max(P_{RowAvg\_StrI}, P_{RowAvg\_StrIII}, P_{RowAvg\_StrV}) = 228.199 \cdot kip$$

Since only Service Limit State I is considered, the controlling average reaction of the piles in a row under Service Limit States $P_{RowAvg\_SerI} := \frac{F_{VFtSerI}}{N_{piles}} + \frac{M_{XFtSerI}}{S_{XX}} = 175.232 \cdot kip$

Tension force in the tension tie on a per-foot basis $T_h := \frac{P_{RowAvg\_Str}}{Spacing_x} \cdot \frac{l_a}{h_a} = 21.942 \cdot \frac{kip}{ft}$

Resistance factor for tension members $\phi_{tension} := 0.9$ **LRFD 5.5.4.2**

Required reinforcing steel area on a per-foot basis $A_{s\_req} := \frac{T_h}{\phi_{tension} \cdot f_y} = 0.406 \cdot \frac{in^2}{ft}$ **LRFD Eq. 5.8.2.4-1**

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**
12 in. for walls and footings greater than 18 in.
For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. **BDG 5.22.01**

Select a spacing for reinforcing steel bars $s_{bar} := 12 \cdot in$

Area of tension steel provided on a per-foot basis $A_{sProvided\_x} := \frac{A_{bar}}{s_{bar}} = 0.6 \cdot \frac{in^2}{ft}$

Check the adequacy of tension tie reinforcement

$$Check := if(A_{sProvided\_x} > A_{s\_req}, "OK", "Not OK") = "OK"$$

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does 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 the Class 1 exposure condition

$$\gamma_e := 1.00$$

LRFD C5.6.7

For large concrete covers, use a 2 in. clear cover.

Distance from extreme tension fiber to the center of the closest flexural reinforcement

$$d_c := 2in + \frac{1}{2} d_{bx} = 2.438 \cdot in$$

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

$$\beta_s := 1 + \frac{d_c}{0.7(t_{footing} - d_c)} = 1.104$$

Next, calculate the tensile stress in the reinforcement at the service limit state, f_{ss}.

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

$$T_{h\_SerI} := \frac{P_{RowAvg\_SerI}}{Spacing_x} \cdot \frac{l_a}{h_a} = 16.849 \cdot \frac{kip}{ft}$$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_{h\_SerI}}{\phi_{tension} \cdot A_{sProvided\_x}} = 31.202 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min(f_{ss1}, 0.6f_y) = 31.202 \cdot ksi$$

Required reinforcing steel bar spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 15.45 \cdot in$$

Check if the spacing provided < the required spacing

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

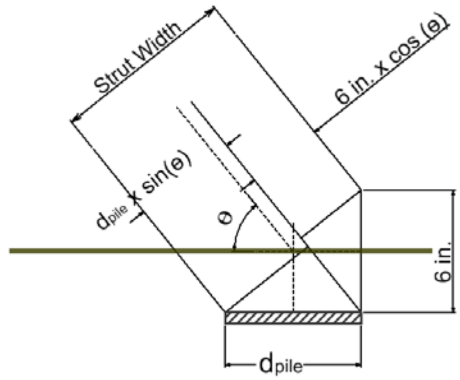
Diagonal Strut Check

The compression force in the diagonal strut is calculated using static equilibrium.

$$P_{uStrut} := \frac{P_{RowAvg\_Str}}{\sin(\theta)} = 287.903 \cdot kip$$

Depth of the selected pile section

$$d_{pile} = 13.61 \cdot in$$



Node 1 is a CCT node at which a tie intersects only from one direction. The surface at which the diagonal strut meets the node is called the strut-to-node interface.

Modification factor to account for confinement, conservatively taken as 1.0 $m := 1$

Resistance factor for the strut $\phi_{strut} := 0.7$ **LRFD 5.5.4.2**

Concrete efficiency factor, assuming crack control reinforcement being present $v_{CCT} := 0.7$ **LRFD Table 5.8.2.5.3a-1**

Width of the strut $w_{strut} := d_{pile} \cdot \sin(\theta) + 6in \cdot \cos(\theta) = 14.446 \cdot in$

Compressive stress at the face of the node where the strut meets the node

$$f_{c\_strut} := \frac{P_{uStrut}}{\phi_{strut} \cdot w_{strut} \cdot Spacing_y} = 0.475 \cdot ksi$$

Limiting compressive stress at the face of the node $f_{cu} := m \cdot v_{CCT} \cdot f_c = 2.1 \cdot ksi$ **LRFD Eq. 5.8.2.5.3a-1**

Check the adequacy of the strut $Check := if(f_{c\_strut} < f_{cu}, "OK", "Not\ OK") = "OK"$

Shrinkage and Temperature Reinforcement Requirement

The following calculations check the adequacy of the flexural reinforcing steel to control shrinkage and temperature stresses in the footing. **LRFD 5.10.6**

Required minimum area of shrinkage and temperature reinforcement

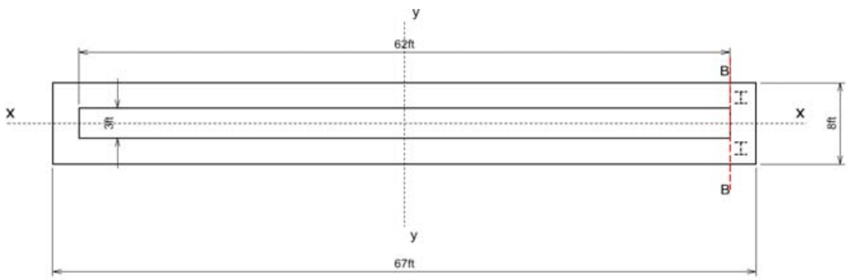
$$A_{shrink.temp} := \min\left[\left[\begin{array}{c} \left(0.60 \frac{in^2}{ft}\right) \\ \max\left[\left[\begin{array}{c} \left(0.11 \frac{in^2}{ft}\right) \\ \left[\dfrac{1.3 \cdot w_{footing} \cdot t_{footing} \cdot \frac{kip}{in \cdot ft}}{2\left(w_{footing} + t_{footing}\right) \cdot f_y}\right] \end{array}\right]\right] \end{array}\right]\right] = 0.284 \cdot \frac{in^2}{ft}$$

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

Therefore, the STM design requires the use of No. 7 bars at 12.0 in. spacing ($A_s = 0.6$ in.^{2}/ft) as the transverse reinforcement at the bottom of the footing.

Longitudinal Reinforcement

The reinforcement design in the longitudinal direction uses the traditional section method. For flexural design of the reinforcement along the longitudinal direction of the footing, the critical section is located at section B-B (at the face of the base wall).



Moment at the Face of the Base Wall (Section B-B)

Distance from the center of the piles in the end column to section B-B

$$Arm := \frac{l_{footing} - l_{wall}}{2} - PileEdgeDist = 1\,ft$$

Moment demand at section B-B on a per-foot basis, Strength I

$$M_{uyStrI} := \frac{Pile_{row}\,P_{uMax} \cdot Arm}{w_{footing}} = 57.982 \cdot \frac{kip \cdot ft}{ft}$$

Moment demand at section B-B on a per-foot basis, Service I

$$M_{uySerI} := \frac{Pile_{row}\,P_{uMax\_SerI} \cdot Arm}{w_{footing}} = 44.997 \cdot \frac{kip \cdot ft}{ft}$$

Note: As per the MDOT practice, the maximum reactions of the two piles at the end column are conservatively assumed to be equal.

Flexural 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.

Select a trial bar size $bar := 7$

Nominal diameter of a reinforcing steel bar $d_{by} := Dia(bar) = 0.875 \cdot in$

Cross-section area of a bar on the flexural tension side $A_{bar} := Area(bar) = 0.6 \cdot in^2$

The spacing of the main reinforcing steel bars in walls and slabs shall not be greater than the lesser of 1.5 times the thickness of the member or 18 in. **LRFD 5.10.3.2**

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. **BDG 5.22.01**

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

Select a spacing for the reinforcing steel bars $s_{bar} := 10 \cdot in$

Provided area of tension steel in a 1-ft wide section

$$A_{sProvided\_y} := \frac{A_{bar} \cdot 12in}{s_{bar}} = 0.72 \cdot in^2$$

Since the moment about the y-axis is smaller than the moment about the x-axis, place the reinforcing bars along the width direction at the bottom of the footing; then place the bars along the length direction directly on top of them.

Effective depth

$$d_{ey} := t_{footing} - Pile\_embd - d_R - \frac{d_{bx}}{2} - \frac{d_{by}}{2} = 26.125 \cdot in$$

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

Select a 1-ft wide strip for the design.

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

Stress block factor $\beta_1 = 0.85$

Solve the following equation of A_s to calculate the required area of steel to satisfy the moment demand. Use an assumed initial A_s value to solve the equation.

Initial assumption $A_s := 1in^2$

Given $$M_{uyStrI} \cdot ft = \phi_f \cdot A_s \cdot f_y \cdot \left[d_{ey} - \frac{1}{2} \cdot \left(\frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot b}\right)\right]$$

Required area of steel $A_{sRequired\_y} := Find(A_s) = 0.503 \cdot in^2$

Check if $A_{sProvided} > A_{sRequired}$

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

Moment capacity of the section with the provided steel

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

$$M_{Provided} = 82.358 \cdot \frac{kip \cdot ft}{ft}$$

Distance from the extreme compression fiber to the neutral axis

$$c := \frac{A_{sProvided\_y} \cdot f_y}{0.85 \cdot f_c \cdot \beta_1 \cdot b} = 1.66 \cdot in$$

Check the validity of assumption, $f_s = f_y$

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

Limits for Reinforcement

LRFD 5.6.3.3

The tensile reinforcement provided must be adequate to develop a factored flexural resistance at least 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 ASTM A615 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 $1.33 \cdot M_{uyStrI} = 77.115 \cdot \frac{kip \cdot ft}{ft}$

The factored flexural resistance required to satisfy the minimum reinforcement requirement $M_{req} := \min(1.33M_{uyStrI}, M_{cr}) = 77.115 \cdot \frac{kip \cdot ft}{ft}$

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

Control of Cracking by Distribution of Reinforcement

LRFD 5.6.7

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. Checking for crack control assures that the actual stress in the reinforcement does 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 the Class 1 exposure condition $\gamma_e := 1.00$

Distance from the extreme tension fiber to the center of the closest flexural reinforcement $d_c := Cover_{ft} = 4 \cdot in$

Ratio of flexural strain at the extreme tension face to the strain at the centroid of the reinforcement layer closest to the tension face $\beta_s := 1 + \frac{d_c}{0.7(t_{footing} - d_c)} = 1.179$

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 distance from the extreme compression fiber to the neutral axis $x := 5 \cdot in$

Given $\frac{1}{2} \cdot b \cdot x^2 = \frac{E_s}{E_c} \cdot A_{sProvided\_y} \cdot (d_{ey} - x)$

Position of the neutral axis $x_{na} := Find(x) = 4.551 \cdot in$

Tensile force in the reinforcing steel due to service limit state moment $T_s := \frac{M_{uySerI}}{d_{ey} - \frac{x_{na}}{3}} \cdot ft = 21.9 \cdot kip$

Stress in the reinforcing steel due to service limit state moment

$$f_{ss1} := \frac{T_s}{A_{sProvided\_y}} = 30.475 \cdot ksi$$

f_{ss} (not to exceed $0.6f_y$)

$$f_{ss} := \min\left(f_{ss1}, 0.6f_y\right) = 30.475 \cdot ksi$$

Required reinforcement bar spacing

$$s_{barRequired} := \frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss}} - 2 \cdot d_c = 11.489 \cdot in$$

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

The required minimum area of shrinkage and temperature reinforcement

$$A_{shrink.temp} = 0.284 \cdot \frac{in^2}{ft}$$

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

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

Therefore, the flexural design requires the use of No. 7 bars at 10.0 in. spacing ($A_s = 0.72$ in.^{2}/ft) as the longitudinal flexural reinforcement at the bottom of the footing.

Design for Shear

One-Way Shear

The STM was used for one-way shear design in the transverse direction. The following calculations present the one-way shear design in the longitudinal direction.

In the longitudinal direction of the footing, the factored shear force at the critical section is computed by calculating the total pile reaction force acting on the footing base that is outside of the critical section.

Depth of equivalent rectangular stress block

$$a := \frac{A_{sProvided\_y} \cdot f_y}{0.85 \cdot f_c \cdot b} = 1.412 \cdot in$$

Effective shear depth

$$d_{vy} := \max\left(d_{ey} - \frac{a}{2}, 0.9 \cdot d_{ey}, 0.72 \cdot t_{footing}\right) = 25.92 \cdot in$$

LRFD 5.7.2.8

Since the piles are located inside the critical sections, there is no need to check one-way shear in the footing length directions.

Two-Way Shear

Critical Perimeter around the Base Wall

Two-way shear (punching shear) in the footing is checked at a critical perimeter around the base wall.

The critical perimeter, b_o, is at a minimum of $0.5d_v$ from the perimeter of the base wall. **LRFD 5.12.8.6.3**

Note: An average effective shear depth d_v is used since the two-way shear area includes both x- and y- directions of the footing.

Effective depth in the transverse direction of the footing

$$d_{ex} := t_{footing} - Pile\_embd - d_R = 27 \cdot in$$

Depth of equivalent rectangular stress block in the transverse direction

$$a_x := \frac{A_{sProvided\_x} \cdot f_y}{0.85 \cdot f_c} = 1.176 \cdot in$$

Effective shear depth in the transverse direction

$$d_{vx} := \max\left(d_{ex} - \frac{a_x}{2}, 0.9 \cdot d_{ex}, 0.72 \cdot t_{footing}\right) = 2.201 \cdot ft$$

Average effective shear depth

$$d_{v\_avg} := \frac{\left(d_{vx} + d_{vy}\right)}{2} = 2.18\ ft$$

All piles are located inside the critical perimeter. Therefore, there is no need to check the two-way shear around the base wall.

Note: The pier in this design example has a base wall. In the absence of a base wall, the two-way shear should be checked around the critical perimeter of a column. The procedure is described in Step 5.10.

Critical Perimeter around a Pile

The critical perimeter around a pile, b_o, is located at a minimum of $0.5d_v$ from the perimeter of the pile. When portions of the critical perimeter are located off the footing, the critical perimeter is limited by the footing edge.

LRFD 5.12.8.6.3

Flange width and depth of the selected pile section

$$b_f = 14.585 \cdot in \qquad d_{pile} = 13.61 \cdot in$$

Check if the critical perimeter is off the footing in y-axis direction

$$OffFooting\_y := if\left(\frac{d_{pile}}{2} + \frac{d_{v\_avg}}{2} > PileEdgeDist, \text{"Yes"}, \text{"No"}\right)$$

$$OffFooting\_y = \text{"Yes"}$$

Check if the critical perimeter is off the footing in x-axis direction

$$OffFooting\_x := if\left(\frac{b_f}{2} + \frac{d_{v\_avg}}{2} > PileEdgeDist, \text{"Yes"}, \text{"No"}\right)$$

$$OffFooting\_x = \text{"Yes"}$$

Side length of the critical perimeter parallel to y-axis

$$b_{0y} := if\left(OffFooting\_y = \text{"Yes"}, \frac{d_{pile}}{2} + \frac{d_{v\_avg}}{2} + PileEdgeDist, d_{pile} + d_{v\_avg}\right)$$

$$b_{0y} = 3.157\ ft$$

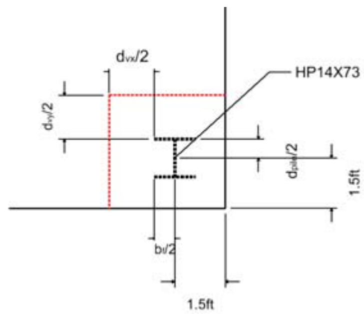
Side length of the critical perimeter parallel to x-axis

$$b_{0x} := if\left(OffFooting\_x = \text{"Yes"}, \frac{b_f}{2} + \frac{d_{v\_avg}}{2} + PileEdgeDist, b_f + d_{v\_avg}\right)$$

$$b_{0x} = 3.198\ ft$$

Critical perimeter

$$b_0 := b_{0x} + b_{0y} = 6.355\ \text{ft}$$



Ratio of long-to-short sides of the pile cross-section

$$\beta_c := \frac{b_f}{d_{pile}} = 1.072$$

Nominal shear resistance

$$V_{n1\_2way} := \left(0.063 + \frac{0.126}{\beta_c}\right)\cdot\sqrt{f_c\cdot ksi}\cdot b_0\cdot d_{v\_avg} = 624.129\cdot kip$$

$$V_{n2\_2way} := 0.126\cdot\left(\sqrt{f_c\cdot ksi}\cdot b_0\cdot d_{v\_avg}\right) = 435.494\cdot kip$$

LRFD Eq. 5.12.8.6.3-1

$$V_{n\_2way} := \min\left(V_{n1\_2way}, V_{n2\_2way}\right) = 435.494\cdot kip$$

Factored shear resistance (capacity)

$$V_{r\_2way} := \phi_v\cdot V_{n\_2way} = 391.945\cdot kip$$

Maximum pile reaction (demand)

$$P_{uMax\_StrI} = 231.926\cdot kip$$

Check if the capacity > the demand

$$Check := if\left(V_{r\_2way} > P_{uMax}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

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**

Longitudinal Direction of the Footing

Available development length

$$l_{dy\_avail} := \frac{l_{footing} - l_{wall}}{2} - Cover_{ft} = 26\cdot in$$

From flexural design

Longitudinal reinforcing steel bar size: No. 7

Bar spacing: 10 in

Assuming that the bars are at high stress, the required bar development length

$$l_{dy.req} := 21in$$

BDG 7.14.01

Check if $l_{dy.avail} > l_{dy.req}$

$$Check := if\left(l_{dy\_avail} > l_{dy.req}, \text{"OK"}, \text{"Not OK"}\right) = \text{"OK"}$$

Transverse Direction of the Footing

Available development length $l_{dx\_avail} := \dfrac{w_{footing} - t_{wall}}{2} - Cover_{ft} = 26 \cdot in$

From flexural design

Longitudinal reinforcing steel bar size: No. 7

Bar spacing: 12 in

Assuming that the bars are at high stress, the required bar development length $l_{dx.req} := 21in$ **BDG 7.14.01**

Check if $l_{dx.avail} > l_{dx.req}$ $Check := if(l_{dx\_avail} > l_{dx.req}, \text{"OK"}, \text{"Not OK"}) = \text{"OK"}$

Shrinkage and Temperature Reinforcement

This requirement for the steel at the bottom of the footing was already checked and satisfied.

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

The spacing of shrinkage and temperature reinforcement shall not exceed the following: **LRFD 5.10.6**

12 in. for walls and footings greater than 18 in.

For all other situations, 3 times the component thickness but not less than 18 in.

Note: MDOT limits reinforcement spacing to a maximum of 18 in. **BDG 5.22.01**

Select a trial bar size $bar := 6$

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

Select a spacing for reinforcing steel bars $s_{barST} := 18 \cdot in$

Provided horizontal reinforcement area $A_{sProvidedST} := \dfrac{A_{barST}}{s_{barST}} = 0.293 \cdot \dfrac{in^2}{ft}$

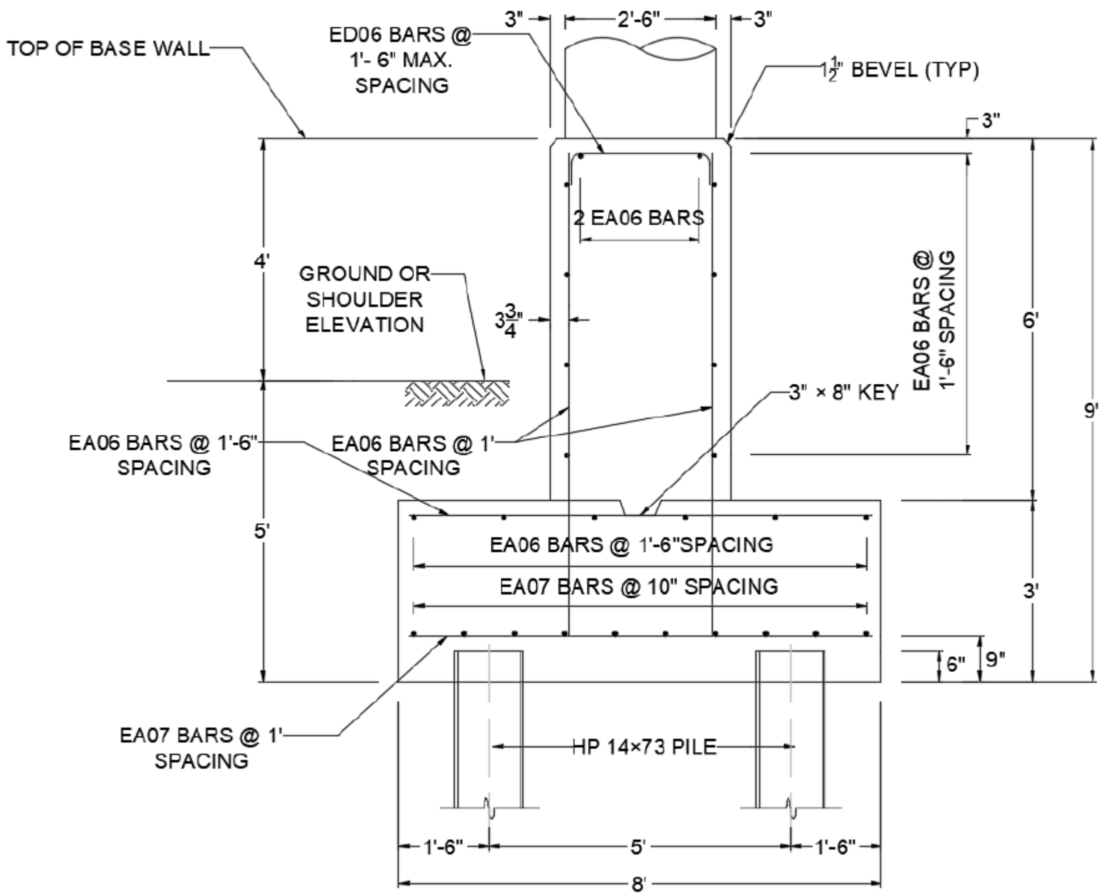
Required shrinkage and temperature steel area (calculated during flexural design) $A_{shrink.temp} = 0.284 \cdot \dfrac{in^2}{ft}$

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

Therefore, the design requires the use of No. 6 bars at 18.0 in. spacing ($A_s = 0.293$ in.^{2}/ft) as the shrinkage and temperature reinforcement at the top of the footing in both longitudinal and transverse directions.

The footing and base wall designs require the following details:

- No. 7 bars @ 12.0 in. spacing (A_s = 0.6 in.^{2}/ft) as the transverse reinforcement at the bottom of the footing
- No. 7 bars @ 10.0 in. spacing (A_s = 0.72 in.^{2}/ft) as the longitudinal flexural reinforcement at the bottom of the footing
- No. 6 bars @ 18.0 in. spacing (A_s = 0.293 in.^{2}/ft) as the shrinkage and temperature reinforcement at the top of the footing in both longitudinal and transverse directions
- Two No. 6 bars at the top of the base wall (BDG 5.22.01)
- No. 6 bars @ 12.0 in. spacing (A_s = 0.44 in.^{2}/ft) on each face of the base wall as the vertical flexural reinforcement
- No. 6 bars @ 18.0 in. spacing (A_s = 0.293 in.^{2}/ft) on each face of the base wall as the horizontal shrinkage and temperature reinforcement.



Note: Certain details are not shown in this drawing for clarity of main reinforcement. Refer to MDOT Bridge Design Guides for additional details.