



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
16 STATE HOUSE STATION
AUGUSTA, MAINE 04333-0016

Janet T. Mills
GOVERNOR

Bruce A. Van Note
COMMISSIONER

January 9, 2023
Subject: Replacement of Changeable
Message Signs
State WIN: 024239.00
Location: **Statewide**
Amendment No. 4

Dear Sir/Ms.:

Make the following changes to the Bid documents:

In the Bid Book:

Insert the attached CMS Calculation book totaling forty-nine pages.

In the plan sheet

Remove page six titled SIGN SPECIAL DETAILS III totaling one page and **Replace** with the attached SIGN SPECIAL DETAILS III totaling one page

The following questions have been received:

Question: Could you please provide more information on the specifications for the cabinet requirements for the CCTV Cameras?

Response: CCTV camera cabinets shall be APX small single door enclosure NEMA 3R DT26 or equivalent.

Question: Could you please provide more information on the specifications for the cabinet requirements for the Changeable Message Signs?

Response: The proposed cabinet should match the existing CMS cabinets. Original specifications required NEMA 4X aluminum enclosures equipped with binding post terminal blocks #2 key lock, and filled backs.

Question: There is a section in CCTV (654.54) for SwRI to assist with configuration of the CCTV devices into Compass. Does SwRI need to assist with configuration of the message boards in Compass?

Response: While SwRI has integrated similar devices, it is the contractor's responsibility to ensure proposed devices can be integrated and to carry any costs necessary for integration.

Consider these changes and information prior to submitting your bid on **January 11, 2023.**

Sincerely,



George M. A. Macdougall P.E.
Contracts & Specifications Engineer

**MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM**

STRUCTURE DESIGN COMPUTATIONS

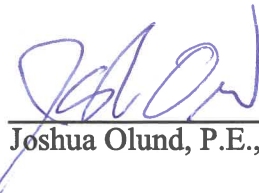
For the Design of:

**Changeable Message Sign Structure Foundation
In
Sidney, Maine**

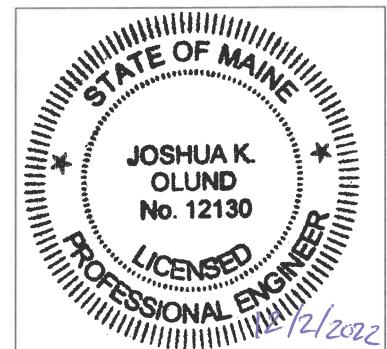
WIN 024239.00

December 2, 2022

*Designed by: Josh McCauley, E.I., HNTB Corporation
Checked by: Dante Guzzi, P.E., HNTB Corporation
In Charge of: Joshua Olund, P.E., PhD, HNTB Corporation*


Joshua Olund, P.E., PhD

12/2/2022
Date



SPECIFICATIONS

DESIGN: Load and Resistance Factor Design per AASHTO LRFD Bridge Design Specifications, Ninth Edition 2020, Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals. First Edition 2015



MaineDOT

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1.0 SUBSTRUCTURE

1.1 LOAD DEVELOPMENT

Sidney CMS Model Load Development

Purpose: To design the changeable message sign support structure located along Interstate 95 Southbound in Sidney Maine.

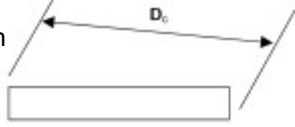
References:

- AASHTO LRFD Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 1st Edition 2015 with Interims through 2020
- MaineDOT Standard Specifications, 2020
- AISC Steel Construction Manual, 14th Edition, 2011
- ASCE 7-16 Minimum Design Loads for Building and Other Structures

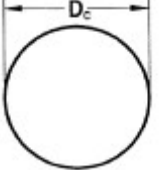
INPUTS

CMS and Sign Support Properties and Dead Loads

CMS

CMS Dead Load:	$DL_{CMS} := 1790 \cdot \text{lbf}$	From NGC Structural Calculations
Longer Dimension of Attached Sign:	$L_{CMS} := (21 \cdot \text{ft} + 1 \text{in})1.25 = 26.354 \text{ft}$	Per Std spec 645.023c
Shorter Dimension of Attached Sign:	$W_{CMS} := (7 \text{ft} + 5 \text{in})1.25 = 9.271 \text{ft}$	D_c and ice load calcs assume CMS span length is greater than the CMS depth
Top of CMS Depth:	$t_{CMS} := 1 \cdot \text{ft} + 4 \text{in}$	
Diameter of Circumscribing Cylinder:	$D_{c_CMS} := \sqrt{W_{CMS}^2 + t_{CMS}^2} = 112.39 \cdot \text{in}$	
CMS Sign Area:	$A_{s_CMS} := L_{CMS} \cdot W_{CMS} = 244.33 \cdot \text{ft}^2$	

Column (HSS 18x0.375)

Column Weight:	$wt_{col} := 70.66 \cdot \text{plf}$	
No. Column Supports:	$no_{col} := 1$	
Column Length:	$L_{col} := 21 \cdot \text{ft} + 4 \text{in}$	Top of footing to top of post
Column Outside Diameter:	$d_{col} := 18 \cdot \text{in}$	
Column Wall Thickness:	$t_{col} := 0.375 \text{in}$	
Column Net Area:	$A_{g_col} := 19.4 \cdot \text{in}^2$	ASCE7-11 Figure 10-1
Diameter of Circumscribing Cylinder:	$D_{c_col} := d_{col} = 18 \cdot \text{in}$	
Column Dead Load:	$DL_{col} := wt_{col} \cdot L_{col} = 1.507 \cdot \text{kip}$	

AISC Table 1-14

Horizontal Bracing (HSS 5.563x0.219) (Use HSS 5.563x0.258)

Vertical Distance between Horizontal Truss Members $D_{truss} := 4\text{ft}$

Truss Chord Weight: $wt_{truss} := 14.63 \cdot \text{plf}$

Truss Chord Length: $L_{truss} := 9.33\text{ft}$ CL of center column to end of horizontal member

Truss Chord Outside Diameter: $d_{truss} := 5.563 \cdot \text{in}$

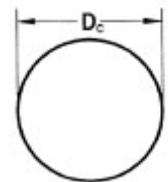
Truss Chord Wall Thickness: $t_{truss} := 0.258\text{in}$

Truss Chord Net Area: $A_{g_truss} := 4.01 \cdot \text{in}^2$

Diameter of Circumscribing Cylinder: $D_{c_truss} := d_{truss} = 5.56 \cdot \text{in}$

Truss Chord Dead Load:
 (one of four members) $DL_{truss} := wt_{truss} \cdot L_{truss} = 0.136 \cdot \text{kip}$

ASCE7-11 Figure 10-1



Cross Bracing (Diagonal and Vertical) (HSS 1.315x0.133)

AISC Table 1-14

Cross Brace Weight: $wt_{cross} := 1.68 \cdot \text{plf}$

Cross Brace Length: $L_{cross} := \sqrt{(D_{truss})^2 + (D_{truss})^2} = 5.657 \cdot \text{ft}$

Diagonal Length
 (conservative)

Cross Brace Outside Diameter: $d_{cross} := 1.315 \cdot \text{in}$

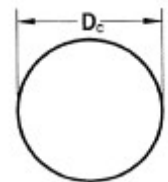
Cross Brace Wall Thickness: $t_{cross} := 0.133\text{in}$

Cross Brace Net Area: $A_{g_cross} := 0.469 \cdot \text{in}^2$

Diameter of Circumscribing Cylinder: $D_{c_cross} := d_{cross} = 1.32 \cdot \text{in}$

Cross Brace Dead Load:
 (single strut) $DL_{cross} := wt_{cross} \cdot L_{cross} = 0.01 \cdot \text{kip}$

ASCE7-11 Figure 10-1



Sign Hanger (W6x9)

Hanger Support Weight:

$$wt_{support} := 9 \cdot plf$$

Hanger Support Length:

$$L_{support} := 7ft + 5in = 7.417 \cdot ft$$

Depth of Support:

$$d_{support} := 5.9in$$

Width of Flange:

$$b_{flg} := 4in$$

Hanger Support Net Area:

$$A_{g_support} := 2.68 \cdot in^2$$

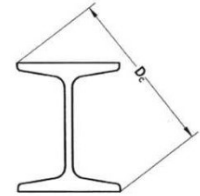
Diameter of Circumscribing Cylinder:

$$D_{c_support} := \sqrt{d_{support}^2 + b_{flg}^2} = 7.13 \cdot in$$

Hanger Support Dead Load:
 (single Hanger Support)

$$DL_{support} := wt_{support} \cdot L_{support} = 0.07 \cdot kip$$

ASCE7-11 Figure 10-1



Sign Hanger (WT5x8.5)

Hanger Weight:

$$wt_{hanger} := 8.5 \cdot plf$$

Hanger Length:

$$L_{hanger} := 7ft + 5in = 7.417 \cdot ft$$

Depth of Hanger:

$$d_{hanger} := 5.06in$$

Width of Flange:

$$b_{hanger_flg} := 4in$$

Hanger Net Area:

$$A_{g_hanger} := 2.50 \cdot in^2$$

Diameter of Circumscribing Cylinder:

$$D_{c_hanger} := \sqrt{d_{hanger}^2 + \left(\frac{b_{hanger_flg}}{2}\right)^2} = 5.44 \cdot in$$

Hanger Support Dead Load:
 (single Hanger Support)

$$DL_{hanger} := wt_{hanger} \cdot L_{hanger} = 0.06 \cdot kip$$

Model Load Development

Ice Load (AASHTO Section 3.7/ASCE7-10 Chapter 10)

Ice Density:	$\gamma_{ice} := 56 \cdot \text{pcf}$	ASCE7-16 10.4.1
Nominal Ice Thickness:	$t := 1.5 \cdot \text{in}$	ASCE7-16 Figure 10.4-2
HeightAbove Ground:	$z := 13.5\text{ft} + W_{CMS} = 22.771 \cdot \text{ft}$	see K_z wind load for calculation
Height Factor:	$f_z := \begin{cases} \left(\frac{z}{33} \right)^{0.10} & \text{if } 0 \cdot \text{ft} < z \leq 900 \cdot \text{ft} \\ 1.4 & \text{if } z > 900 \cdot \text{ft} \end{cases} = 0.964$	ASCE7-16 EQ. 10.4-4
Importance Factor:	$I_i := 0.8$	ASCE7-16 Table 1.5-2, Risk Category I
Topographic Factor:	$K_{zt} := 1.0$	ASCE7-16 26.8.2
Design Ice Thickness:	$t_d := t \cdot I_i \cdot f_z \cdot (K_{zt})^{0.35} = 1.156 \cdot \text{in}$	ASCE7-16 10.4.6

Sign Support Column

Column Ice Weight:	$A_{i_col} := \pi \cdot t_d \cdot (D_{c_col} + t_d) = 69.59 \cdot \text{in}^2$	ASCE7-16 EQ. 10.4-1
Column Ice Load:	$W_{t_{ice_col}} := \gamma_{ice} \cdot A_{i_col} = 27.06 \cdot \text{plf}$	

Sign Support Truss Chords

Chord Bracing Ice Area:	$A_{i_chord} := \pi \cdot t_d \cdot (D_{c_truss} + t_d) = 24.41 \cdot \text{in}^2$	ASCE7-16 EQ. 10.4-1
Chord Bracing Ice Load:	$W_{t_{ice_chord}} := \gamma_{ice} \cdot A_{i_chord} = 9.49 \cdot \text{plf}$	

Sign Support Cross Bracing

Cross Bracing Ice Area:	$A_{i_cross} := \pi \cdot t_d \cdot (D_{c_cross} + t_d) = 8.98 \cdot \text{in}^2$	ASCE7-16 EQ. 10.4-1
Cross Bracing Ice Load:	$W_{t_{ice_cross}} := \gamma_{ice} \cdot A_{i_cross} = 3.49 \cdot \text{plf}$	

Hanger Support

Hanger Support Ice Area: $A_{i_support} := \pi \cdot t_d \cdot (D_{c_support} + t_d) = 30.09 \cdot \text{in}^2$ ASCE7-16 EQ. 10.4-1

Hanger Support Ice Load: $W_{t_ice_support} := \gamma_{ice} \cdot A_{i_support} = 11.7 \cdot \text{plf}$

Hanger

Hanger Ice Area: $A_{i_hanger} := \pi \cdot t_d \cdot (D_{c_hanger} + t_d) = 23.97 \cdot \text{in}^2$ ASCE7-16 EQ. 10.4-1

Hanger Ice Load: $W_{t_ice_hanger} := \gamma_{ice} \cdot A_{i_hanger} = 9.32 \cdot \text{plf}$

CMS Sign

Sign Ice Volume: $V_{i_sign} := 0.8\pi \cdot t_d \cdot A_{s_CMS} = 59.17 \cdot \text{ft}^3$ ASCE7-16 EQ. 10.4-2

Sign Ice Load: $I_{L_sign_1} := \gamma_{ice} \cdot V_{i_sign} = 3313.47 \cdot \text{lbft}$

Wind Load (AASHTO Section 3.8)

Note: Assume the sign covers the entire truss, therefor no need to calculate wind load on truss or chord members.

Basic Wind Speed: $V_{ww} := 120 \cdot \text{mph}$ AASHTO Figure 3.8-2b (1700 year MRI)

Elevated Locations and Special Wind Regions: *Not applicable per AASHTO 3.8.2.1 and 3.8.3*

Height and Exposure Factor: $K_z := 0.90 + \frac{(z - 20\text{ft})(0.98 - 0.90)}{(30\text{ft} - 20\text{ft})} = 0.922$ AASHTO Table C3.8.4-1, 30 ft Sign Structure

Directionality Factor: $K_d := 0.85$ AASHTO Table 3.8.5-1, for Overhead Truss and Support

Gust Effect Factor: $G_{ww} := 1.14$ AASHTO 3.8.6

Wind Drag Coefficient (CMS): $C_{d_CMS} := 1.7$ AASHTO Table 3.8.7-1, Dynamic Message Signs

Wind Drag Coefficient (Columns): $C_{d_col} := \begin{cases} 1.10 & \text{if } 0.8V \cdot d_{col} \leq 39 \cdot \text{mph} \cdot \text{ft} \\ 0.45 & \text{if } 0.8V \cdot d_{col} \geq 78 \cdot \text{mph} \cdot \text{ft} \\ \frac{129}{\left(\frac{0.8V \cdot d_{col}}{\text{mph} \cdot \text{ft}}\right)^{1.3}} & \text{otherwise} \end{cases}$ $C_{d_col} = 0.45$ AASHTO Table 3.8.7-1

Wind Pressure: *Excludes drag coefficient Cd, which is applied conditionally based on member properties* AASHTO Eq. 3.8.1-1

$$P_z := 0.00256 K_z \cdot K_d \cdot G \cdot V^2 \cdot \left(\frac{\text{psf}}{\text{mph}^2} \right) = 32.94 \cdot \text{psf}$$

Sign Support Columns

Column Wind Load: $WL_{col} := P_z \cdot C_{d_col} \cdot d_{col} = 22.24 \cdot \text{plf}$

Sign Panel

Sign Panel Wind Load: $WL_{sign} := P_z \cdot C_{d_CMS} \cdot A_{s_CMS} = 13.68 \cdot \text{kip}$

Vertical Force Summary (Axial, y)

Dead Loads

Columns: $DL_{col} := (wt_{col} \cdot L_{col}) \cdot 1.05 = 1.58 \cdot \text{kip}$ *Add 5% for miscellaneous*

Truss: $DL_{truss} := (4 \cdot wt_{truss} \cdot L_{truss}) = 0.55 \cdot \text{kip}$ *4 chords total*

Cross Bracing: $DL_{cross} := (10 \cdot wt_{cross} \cdot L_{cross}) = 0.1 \cdot \text{kip}$ *10 braces total*

Hanger Support: $DL_{support} := (5 \cdot wt_{support} \cdot L_{support}) = 0.33 \cdot \text{kip}$ *5 supports total*

Hanger: $DL_{hanger} := (5 \cdot wt_{hanger} \cdot L_{hanger}) = 0.32 \cdot \text{kip}$ *5 supports total*

Sign: $DL_{sign} := DL_{CMS} = 1.79 \cdot \text{kip}$

Total: $P_{DL_ped} := DL_{col} + DL_{truss} + DL_{cross} + DL_{support} + DL_{hanger} + DL_{sign} = 4.66 \cdot \text{kip}$

Ice Loads

Columns:	$IL_{col} := wt_{ice_col} \cdot L_{col} = 0.58 \cdot kip$	
Truss:	$IL_{truss} := (4 \cdot wt_{ice_chord} \cdot L_{truss}) = 0.35 \cdot kip$	<i>4 chords total</i>
Cross Bracing:	$IL_{cross} := (10 \cdot wt_{ice_cross} \cdot L_{cross}) = 0.2 \cdot kip$	<i>10 braces total</i>
Hanger Support:	$IL_{support} := (5 \cdot wt_{ice_support} \cdot L_{support}) = 0.43 \cdot kip$	<i>5 supports total</i>
Hanger:	$IL_{hanger} := (5 \cdot wt_{ice_hanger} \cdot L_{hanger}) = 0.35 \cdot kip$	<i>5 supports total</i>
Sign:	$IL_{sign} := IL_{sign_1} = 3.31 \cdot kip$	
Total:	$P_{ILped} := IL_{col} + IL_{truss} + IL_{cross} + IL_{support} + IL_{hanger} + IL_{sign} = 5.22 \cdot kip$	

Shear z-axis Summary

Longitudinal Wind Load

Columns:	$V_{WLz_col} := WL_{col} \cdot L_{col} = 0.47 \cdot kip$
Sign:	$V_{WLz_sign} := WL_{sign} = 13.68 \cdot kip$
Total:	$V_{WLz_ped} := V_{WLz_col} + V_{WLz_sign} = 14.16 \cdot kip$

Moment x-axis Summary

Longitudinal Wind Load

Columns: $M_{WLx_col} := \left(WL_{col} \cdot \frac{L_{col}^2}{2} \right) = 5.06 \cdot \text{kip} \cdot \text{ft}$

Sign: $M_{WLx_sign} := WL_{sign} \cdot L_{col} = 291.88 \cdot \text{kip} \cdot \text{ft}$

Total: $M_{WLx_ped} := M_{WLx_col} + M_{WLx_sign} = 296.94 \cdot \text{kip} \cdot \text{ft}$

Shear x-axis Summary

Transverse Wind Load

Columns: $V_{WLx_col} := (WL_{col} \cdot L_{col}) = 0.47 \cdot \text{kip}$

Sign: $V_{WLx_sign} := P_z \cdot C_{d_CMS} \cdot (W_{CMS} \cdot t_{CMS}) = 0.69 \cdot \text{kip}$

Total: $V_{WLx_ped} := V_{WLx_col} + V_{WLx_sign} = 1.17 \cdot \text{kip}$

Moment y-axis Summary

Torsional Wind Load

CMS effective moment arm for torsional column effects due to the sign (longitudinal):

$CMS_{Offset.long} := L_{CMS} \cdot 0.15 = 3.953 \cdot \text{ft}$ AASHTO 3.9.4.2

CMS Normal Torsion Moment: (AASHTO 3.9.4.2):

$M_{WLy_ped} := WL_{sign} \cdot CMS_{Offset.long} = 54087 \cdot \text{ft} \cdot \text{lbf}$

Moment z-axis Summary

Transverse Wind Load

Columns: $M_{WLz_col} := \left(WL_{col} \cdot \frac{L_{col}^2}{2} \right) = 5.06 \cdot \text{kip} \cdot \text{ft}$

Sign: $M_{WLz_sign} := P_z \cdot C_{d_CMS} \cdot (W_{CMS} \cdot t_{CMS}) \cdot L_{col} = 14.77 \cdot \text{kip} \cdot \text{ft}$

Total: $M_{WLz_ped} := M_{WLz_col} + M_{WLz_sign} = 19.83 \cdot \text{kip} \cdot \text{ft}$

Service Wind Moment and Shears

Service wind speeds are based off AASHTO Figure 3.8-4b per AASHTO Table 3.4-1.

Basic Wind Speed: $V_{serv} := 76 \cdot \text{mph}$ AASHTO Figure 3.8-4b (10 year MRI)

Wind Pressure: Excludes drag coefficient C_d , which is applied conditionally based on member properties AASHTO Eq. 3.8.1-1

$$P_{z.serv} := 0.00256 K_z \cdot K_d \cdot G \cdot V_{serv}^2 \cdot \left(\frac{\text{psf}}{\text{mph}^2} \right) = 13.21 \cdot \text{psf}$$

Sign Support Columns

Column Wind Load (Service): $WL_{col.serv} := P_{z.serv} \cdot C_{d_col} \cdot d_{col} = 8.92 \cdot \text{plf}$

Sign Panel

Sign Panel Wind Load (Service): $WL_{sign.serv} := P_{z.serv} \cdot C_{d_CMS} \cdot A_{s_CMS} = 5.49 \cdot \text{kip}$

Shear z-axis Summary (Service)

Longitudinal Wind Load

Columns: $V_{WLz_col.serv} := WL_{col.serv} \cdot L_{col} = 0.19 \cdot \text{kip}$

Sign: $V_{WLz_sign.serv} := WL_{sign.serv} = 5.49 \cdot \text{kip}$

Total: $V_{WLz_ped.serv} := V_{WLz_col.serv} + V_{WLz_sign.serv} = 5.68 \cdot \text{kip}$

Moment x-axis Summary (Service)

Longitudinal Wind Load

Columns: $M_{WLx_col.serv} := \left(WL_{col.serv} \cdot \frac{L_{col}^2}{2} \right) = 2.03 \cdot \text{kip} \cdot \text{ft}$

Sign: $M_{WLx_sign.serv} := WL_{sign.serv} \cdot L_{col} = 117.08 \cdot \text{kip} \cdot \text{ft}$

Total: $M_{WLx_ped.serv} := M_{WLx_col.serv} + M_{WLx_sign.serv} = 119.11 \cdot \text{kip} \cdot \text{ft}$

Summary of Pedestal Forces

Dead Load

$P_{DL_ped} = 4.663 \cdot \text{kip}$ Axial

Ice Load

$P_{IL_ped} = 5.222 \cdot \text{kip}$ Axial

Wind Load

$V_{WLz_ped} = 14.156 \cdot \text{kip}$ Longitudinal Shear

$M_{WLx_ped} = 297 \cdot \text{kip} \cdot \text{ft}$ Longitudinal Moment

$V_{WLx_ped} = 1.167 \cdot \text{kip}$ Transverse Shear

$M_{WLz_ped} = 20 \cdot \text{kip} \cdot \text{ft}$ Transverse Moment

$M_{WLy_ped} = 54 \cdot \text{kip} \cdot \text{ft}$ Torsional Moment

Wind Load (Service)

$V_{WLz_ped.serv} = 5.678 \cdot \text{kip}$ Longitudinal Shear

$M_{WLx_ped.serv} = 119 \cdot \text{kip} \cdot \text{ft}$ Longitudinal Moment

1.2 FOUNDATION DESIGN

Spread Footing Design

Purpose To design the spread footing foundation alternative.

Notes and Assumptions: Forces were calculated in a separate load development worksheet. All loads were determined using AASHTO for Structural Supports for Highway Signs Specifications and All capacities were determined using AASHTO Bridge Design Specifications

References

- AASHTO LRFD Standard Specification for Structural Supports for Highway Signs, Luminaries, and Traffic Signals, 1st Edition 2015 (referred to as AASHTO) with Interims through 2020
- AASHTO LRFD Bridge Design Specifications, 9th Edition with 2016 Interims (referred to as AASHTO LRFD)
- AISC Steel Construction Manual, 14th Edition, 2011
- MacGregor, J.G., 2009, "Reinforced Concrete: Mechanics and Design, 5th Edition"
- Geotechnical Report provided by Schonewald Engineering Associates, dated 11/28/2022

INPUTS

Resistance Factors

$\phi_{v.str} := 0.90$	Shear, Strength Load Cases (AASHTO LRFD 5.5.4.2.1)
$\phi_{v.ser} := 1.0$	Shear, Extreme Event and Service Load Cases
$\phi_{bear.str} := 0.45$	Bearing for strength limit state (per Geotech Report)
$\phi_{bear.ser} := 1.0$	Bearing for service and extreme limit state (per Geotech Report)
$\phi_{bear.ext} := 1.0$	Bearing for service and extreme limit state (per Geotech Report)
$\phi_{slide.str} := 0.8$	Sliding resistance between soil and foundation (AASHTO LRFD Table 10.5.5.2.2-1) (Strength)
$\phi_{slide.ser} := 1.0$	Shear resistance between soil and foundation (AASHTO LRFD Table 10.5.5.2.2-1) (Service)
$\phi_{slide.ext} := 1.0$	Shear resistance between soil and foundation (AASHTO LRFD Table 10.5.5.2.2-1) (Extreme)

Load Factors

$\gamma_{STR1.DC} := 1.25$	Strength I DC (AASHTO Table 3.4-1)
$\gamma_{EXT1.max.DC} := 1.10$	Extreme I DC(AASHTO Table 3.4-1)
$\gamma_{EXT1.min.DC} := 0.9$	Extreme I DC(AASHTO Table 3.4-1)
$\gamma_{STR1.EV} := 1.35$	Strength I Vertical Earth (AASHTO Table 3.4-1)
$\gamma_{EXT1.max.EV} := 1.35$	Extreme I Vertical Earth (AASHTO Table 3.4-1)
$\gamma_{EXT1.min.EV} := 1.0$	Extreme I Vertical Earth (AASHTO Table 3.4-1)
$\gamma_{EXT1.W} := 1.0$	Extreme I Wind Load (AASHTO Table 3.4-1)

$\gamma_{SER1_DC} := 1.0$

Extreme I DC (AASHTO Table 3.4-1)

$\gamma_{SER1_EV} := 1.0$

Extreme I EV (AASHTO Table 3.4-1)

$\gamma_{SER1_W} := 1.0$

Extreme I Wind Load (AASHTO Table 3.4-1)

$\gamma_{EXT1_I} := 1.0$

Extreme I Ice Component (AASHTO Table 3.4-1)

Strength I	Extreme Event 1 - Wind Case 1 - Max Vertical Load	Extreme Event 1 - Wind Case 1 - Min Vertical Load	Extreme Event 1 - Wind Case 2 - Max Vertical Load	Extreme Event 1 - Wind Case 2 - Min Vertical Load	Extreme Event 1 - Wind Case 3 - Max Vertical Load	Extreme Event 1 - Wind Case 3 - Min Vertical Load
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$$\gamma_{str_ext} := \begin{pmatrix} \gamma_{STR1_DC} & \gamma_{EXT1.max_DC} & \gamma_{EXT1.min_DC} & \gamma_{EXT1.max_DC} & \gamma_{EXT1.min_DC} & \gamma_{EXT1.max_DC} & \gamma_{EXT1.min_DC} \\ \gamma_{STR1_EV} & \gamma_{EXT1.max_EV} & \gamma_{EXT1.min_EV} & \gamma_{EXT1.max_EV} & \gamma_{EXT1.min_EV} & \gamma_{EXT1.max_EV} & \gamma_{EXT1.min_EV} \\ 0 & \gamma_{EXT1_I} & 0 & \gamma_{EXT1_I} & 0 & \gamma_{EXT1_I} & 0 \\ 0 & \gamma_{EXT1_W} & \gamma_{EXT1_W} & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & \gamma_{EXT1_W} & \gamma_{EXT1_W} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & \gamma_{EXT1_W} & \gamma_{EXT1_W} \end{pmatrix}$$

Service 1 - Wind Case 1	Service 1 - Wind Case 2	Service 1 - Wind Case 3
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$$\gamma_{ser} := \begin{pmatrix} \gamma_{SER1_DC} & \gamma_{SER1_DC} & \gamma_{SER1_DC} \\ \gamma_{SER1_EV} & \gamma_{SER1_EV} & \gamma_{SER1_EV} \\ 0 & 0 & 0 \\ \gamma_{SER1_W} & 0 & 0 \\ 0 & \gamma_{SER1_W} & 0 \\ 0 & 0 & \gamma_{SER1_W} \end{pmatrix}$$

Dead Load	
Earth Vertical	
Ice Load	
Wind Load (Case 1)	
Wind Load (Case 2)	
Wind Load (Case 3)	

$\gamma := \text{augment}(\gamma_{str_ext}, \gamma_{ser})$

$$\gamma = \begin{pmatrix} 1.25 & 1.1 & 0.9 & 1.1 & 0.9 & 1.1 & 0.9 & 1 & 1 & 1 \\ 1.35 & 1.35 & 1 & 1.35 & 1 & 1.35 & 1 & 1 & 1 & 1 \\ 0 & 1 & 0 & 1 & 0 & 1 & 0 & 0 & 0 & 0 \\ 0 & 1 & 1 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & 1 & 1 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 & 1 & 0 & 0 & 1 \end{pmatrix}$$

$i := 0..9$	Counter for all load cases
$i_{str} := 0$	Counter corresponding to last strength limit load cases
$i_{EE} := 6$	Counter corresponding to last extreme event limit load cases
$i_{ser} := 9$	Counter corresponding to last service limit load cases

Foundation Geometry and Inputs

Footing Dimensions

$L_{foot} := 16ft$	Longitudinal Length of Footing
$B_{foot} := 8ft$	Transverse Length of Footing
$t_{foot} := 1.5ft$	Thickness of Footing
$A_{foot} := B_{foot} \cdot L_{foot} = 128ft^2$	Area of Footing
$H_{ped} := 4.5ft$	Height of Footing Pedestal
$L_{ped} := 3ft$	Length of footing pedestal
$W_{ped} := 3ft$	Width of footing pedestal
$A_{ped} := L_{ped} \cdot W_{ped} = 9ft^2$	Area of Pedestal

Concrete Properties

$\gamma_{conc} := 150pcf$	Unit weight of concrete
$f_y := 60ksi$	Yield Strength of Reinforcement
$f'_c := 5ksi$	Concrete Compressive Strength (Concrete Class LP)
$E_s := 29000ksi$	Elastic modulus of the reinforcing steel
$E_c := 1820ksi \sqrt{\frac{f'_c}{ksi}} = 4070 \cdot ksi$	Elastic Modulus of the Concrete (AASHTO LRFD C5.4.2.4-3)
$n := \frac{E_s}{E_c} = 7.126$	Modular Ratio
$f_r := 0.24ksi \sqrt{\frac{f'_c}{ksi}} = 536.656 \text{ psi}$	Modulus of rupture (AASHTO LRFD 5.4.2.6)
$\gamma_1 := 1.6$	Flexural cracking variability factor (AASHTO LRFD 5.6.3.3)
$\gamma_3 := 0.67$	Ratio of yield strength to ultimate strength of rebar (AASHTO LRFD 5.6.3.3)
$\gamma_e := 1.0$	Exposure Factor - Class 1 (AASHTO LRFD 5.6.7)

Max_Aggregate_Size := 1.5in

Maximum size of the coarse aggregate (LP Concrete MaineDOT Std. Specs)

clr_bot := 3in

Clear cover from bottom of footing

clr_side := 3in

Clear cover from all sides except bottom of footing

D_bar.main := $\frac{8}{8}$ in

Diameter of reinforcing bars in the main reinforcement in the footing (Bottom longitudinal)

D_bar.sec := $\frac{4}{8}$ in

Diameter of reinforcing bars in the secondary reinforcement in the footing (Top longitudinal and both mats transverse)

S_bot.long := 12in

Bar spacing of bottom bars in the longitudinal direction.

S_top.long := 12in

Bar spacing of top bars in the longitudinal direction.

S_bot.trans := 12in

Bar spacing of bottom bars in the transverse direction.

S_top.trans := 12in

Bar spacing of top bars in the transverse direction.

Soil Properties

$\delta_{\text{soil.conc}}$:= 26deg

soil-concrete interface friction angle (per Geotech Report)

γ_{soil} := 135pcf

unit weight of soil

ϕ_f := 36deg

Internal Friction Angle of Soil (MeDOT BDG Table 3-3)

$q_{n.ser}$:= 3ksf

Nominal bearing resistance for Service (per Geotech Report)

$q_{n.str} := \frac{2.3ksf}{\phi_{bear_str}} = 5.111 \cdot ksf$

Nominal bearing resistance for Strength (per Geotech Report, given factored resistance)

Loads

Sign Structure Dead Load (unfactored)

P_DL := 4.663kip

Note: all loads are applied at the top of the pedestal

Axial Dead Load from Column

V_x.DL := 0kip

Longitudinal Shear Dead Load from Column (at base plate)

V_y.DL := 0kip

Transverse Shear Dead Load from Column (at base plate)

M_x.DL := 0.00kip-ft

Longitudinal Bending Dead Load from Column (at base plate)

M_y.DL := 0kip-in

Transverse Bending Dead Load from Column (at base plate)

Sign Structure Ice Load (unfactored)

$P_{IL} := 5.222\text{kip}$

Axial Ice Load from Column

$V_{x,IL} := 0\text{kip}$

Longitudinal Shear Ice Load from Column (at base plate)

$V_{y,IL} := 0\text{kip}$

Transverse Shear Ice Load from Column (at base plate)

$M_{x,IL} := 0.00\text{kip-ft}$

Longitudinal Bending Ice Load from Column (at base plate)

$M_{y,IL} := 0\text{kip-in}$

Transverse Bending Ice Load from Column (at base plate)

Sign Structure Longitudinal Wind Load (unfactored)

$P_{WLx} := 0\text{kip}$

Column Axial Force from Longitudinal Wind Load

$V_{x,WLx} := 14.156\text{kip}$

Column Longitudinal Shear Force from Longitudinal Wind Load

$V_{x,WLx, serv} := 5.678\text{kip}$

Column Longitudinal Shear Force from Longitudinal Wind Load (Service)

$V_{y,WLx} := 0\text{kip}$

Column Transverse Shear Force from Longitudinal Wind Load

$M_{x,WLx} := 297\text{kip-ft}$

Column Longitudinal Bending Force Load from Longitudinal Wind Load

$M_{x,WLx, serv} := 119\text{kip-ft}$

Column Longitudinal Bending Force Load from Longitudinal Wind Load (Service)

$M_{y,WLx} := 0\text{kip-in}$

Column Transverse Bending Force from Longitudinal Wind Load

$M_{z,WLx} := 54\text{kip-ft}$

Column Torsional Moment from Longitudinal Wind Load

Sign Structure Transverse Wind Load (unfactored)

$P_{WLy} := 0\text{kip}$

Column Axial Force from Transverse Wind Load

$V_{x,WLy} := 0\text{kip}$

Column Longitudinal Shear Force from Transverse Wind Load

$V_{y,WLy} := 1.167\text{kip}$

Column Transverse Shear Force from Transverse Wind Load

$M_{x,WLy} := 0\text{kip-in}$

Column Longitudinal Bending Force Load from Transverse Wind Load

$M_{y,WLy} := 20\text{kip-ft}$

Column Transverse Bending Force from Transverse Wind Load

Self Weight Forces (unfactored)

$$P_{\text{foot}} := A_{\text{foot}} \cdot t_{\text{foot}} \cdot \gamma_{\text{conc}} = 28.8 \cdot \text{kip}$$

Footing Weight

$$P_{\text{ped}} := A_{\text{ped}} \cdot H_{\text{ped}} \cdot \gamma_{\text{conc}} = 6.075 \cdot \text{kip}$$

Pedestal Weight

$$P_{\text{soil}} := (A_{\text{foot}} - A_{\text{ped}}) \cdot (H_{\text{ped}}) \cdot \gamma_{\text{soil}} = 72.292 \cdot \text{kip}$$

Weight of Soil on Footing

Summary of Unfactored Loads (at base of footing)

Note: add soil weight as dead load on the footing only, Moments are taken about the base of the footing

$$P_{\text{max.unfact}} := \begin{bmatrix} P_{\text{DL}} + P_{\text{foot}} + P_{\text{ped}} \\ P_{\text{soil}} \\ P_{\text{IL}} \\ P_{\text{WLx}} \\ P_{\text{WLy}} \\ 0.75 \cdot (P_{\text{WLx}} + P_{\text{WLy}}) \end{bmatrix} = \begin{pmatrix} 39.538 \\ 72.292 \\ 5.222 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot \text{kip}$$

Dead Load
Earth Load
Ice Load
Wind Load (Case 1)
Wind Load (Case 2)
Wind Load (Case 3)

Unfactored Axial Loads

$$V_{\text{x.max.unfact}} := \begin{bmatrix} V_{\text{x.DL}} \\ 0 \\ V_{\text{x.IL}} \\ V_{\text{x.WLx}} \\ V_{\text{x.WLy}} \\ 0.75 \cdot (V_{\text{x.WLx}} + V_{\text{x.WLy}}) \end{bmatrix} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 14.156 \\ 0 \\ 10.617 \end{pmatrix} \cdot \text{kip}$$

Unfactored Longitudinal Shear Loads

$$V_{\text{y.max.unfact}} := \begin{bmatrix} V_{\text{y.DL}} \\ 0 \\ V_{\text{y.IL}} \\ V_{\text{y.WLx}} \\ V_{\text{y.WLy}} \\ 0.75 \cdot (V_{\text{y.WLx}} + V_{\text{y.WLy}}) \end{bmatrix} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 1.167 \\ 0.875 \end{pmatrix} \cdot \text{kip}$$

Unfactored Transverse Shear Loads

$$M_{\text{x.max.unfact}} := \begin{bmatrix} M_{\text{x.DL}} \\ 0 \\ M_{\text{x.IL}} \\ M_{\text{x.WLx}} \\ M_{\text{x.WLy}} \\ 0.75 \cdot (M_{\text{x.WLx}} + M_{\text{x.WLy}}) \end{bmatrix} + V_{\text{x.max.unfact}} \cdot (H_{\text{ped}} + t_{\text{foot}}) = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 381.936 \\ 0 \\ 286.452 \end{pmatrix} \text{ft} \cdot \text{kip}$$

Unfactored Longitudinal Bending Loads at base of pedestal

$$M_{y,max.unfact} := \begin{bmatrix} M_{y,DL} \\ 0 \\ M_{y,IL} \\ M_{y,WLx} \\ M_{y,WLy} \\ 0.75 \cdot (M_{y,WLx} + M_{y,WLy}) \end{bmatrix} + V_{y,max.unfact} (H_{ped} + t_{foot}) = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 27.002 \\ 20.252 \end{pmatrix} \text{ ft-kip}$$

Unfactored Transverse Bending Loads
 at base of pedestal

Service level moments are included separately for now, and will be merged into the factored vectors below. Conservatively keep the moment due to wind transverse to the sign based on the extreme event wind speeds.

$$M_{x,ser.unfact} := \begin{bmatrix} M_{x,WLx.serv} \\ 0 \\ 0.75 \cdot (M_{x,WLx.serv}) \end{bmatrix} + \begin{pmatrix} V_{x,WLx.serv} \\ 0 \\ 0.75 \cdot V_{x,WLx.serv} \end{pmatrix} (H_{ped} + t_{foot}) = \begin{pmatrix} 153.068 \\ 0 \\ 114.801 \end{pmatrix} \text{ ft-kip}$$

Summary of Factored Loads

$$P_u := \gamma^T \cdot P_{max.unfact} \quad P_u = \begin{pmatrix} 147.017 \\ 146.309 \\ 107.877 \\ 146.309 \\ 107.877 \\ 146.309 \\ 107.877 \\ 111.831 \\ 111.831 \\ 111.831 \end{pmatrix} \cdot \text{kip}$$

Strength I	
Extreme Event 1 - Wind Case 1 - Max Vertical	
Extreme Event 1 - Wind Case 1 - Min Vertical	
Extreme Event 1 - Wind Case 2 - Max Vertical	
Extreme Event 1 - Wind Case 1 - Min Vertical	
Extreme Event 1 - Wind Case 3 - Max Vertical	
Extreme Event 1 - Wind Case 1 - max Vertical	
Service 1 - Wind Case 1	
Service 1 - Wind Case 2	
Service 1 - Wind Case 3	

Factored axial loads

$$V_{x,u} := \gamma^T \cdot V_{x,max.unfact} = \begin{pmatrix} 0 \\ 14.156 \\ 14.156 \\ 0 \\ 0 \\ 10.617 \\ 10.617 \\ 14.156 \\ 0 \\ 10.617 \end{pmatrix} \cdot \text{kip}$$

Factored longitudinal shear loads

$$V_{y,u} := \gamma^T \cdot V_{y,max.unfact} \quad V_{y,u} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 1.167 \\ 1.167 \\ 0.875 \\ 0.875 \\ 0 \\ 1.167 \\ 0.875 \end{pmatrix} \text{ kip} \quad \text{Factored transverse shear loads}$$

$$M_{x,u} := \gamma^T \cdot M_{x,max.unfact} = \begin{pmatrix} 0 \\ 381.936 \\ 381.936 \\ 0 \\ 0 \\ 286.452 \\ 286.452 \\ 381.936 \\ 0 \\ 286.452 \end{pmatrix} \text{ ft-kip} \quad \text{Factored longitudinal bending loads (note this is overwritten to correct service wind loads below)}$$

$$M_{y,u} := \gamma^T \cdot M_{y,max.unfact} \quad M_{y,u} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 27.002 \\ 27.002 \\ 20.252 \\ 20.252 \\ 0 \\ 27.002 \\ 20.252 \end{pmatrix} \text{ kip-ft} \quad \text{Factored transverse bending loads}$$

Add in Service Moments due to longitudinal wind

$$M_{x,u.ser} := \gamma^{SER1_W} \cdot M_{x.ser.unfact} = \begin{pmatrix} 153.068 \\ 0 \\ 114.801 \end{pmatrix} \text{ ft-kip}$$

$$M_{x,u} := \text{stack}\left[\text{submatrix}(M_{x,u}, 0, 6, 0, 0), M_{x,u.ser}\right] \quad M_{x,u} = \begin{pmatrix} 0 \\ 381.936 \\ 381.936 \\ 0 \\ 0 \\ 286.452 \\ 286.452 \\ 153.068 \\ 0 \\ 114.801 \end{pmatrix} \text{ ft-kip}$$

Check Bearing Pressure

$$e_x := \frac{M_{x,u}}{(P_u)} = \begin{pmatrix} 0 \\ 2.61 \\ 3.54 \\ 0 \\ 0 \\ 1.958 \\ 2.655 \\ 1.369 \\ 0 \\ 1.027 \end{pmatrix} \text{ ft}$$

Eccentricity of Axial Load in Longitudinal Loading

$$e_y := \frac{M_{y,u}}{(P_u)} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0.185 \\ 0.25 \\ 0.138 \\ 0.188 \\ 0 \\ 0.241 \\ 0.181 \end{pmatrix} \text{ ft}$$

Eccentricity of Axial Load in Transverse Loading

$$\text{Check}_{\text{eccen}} := \left(\text{if} \left(\max(e_x) < \frac{L_{\text{foot}}}{3} \wedge \max(e_y) < \frac{B_{\text{foot}}}{3}, \text{"Resultant Eccentricity OK"}, \text{"Increase Size of Footing"} \right) \right)$$

Check_{eccen} = "Resultant Eccentricity OK"

Check AASHTO LRFD 10.6.3.3 Eccentricity Limits

$$\sigma_{v_i} := \frac{P_{u_i}}{(B_{\text{foot}} - 2 \cdot e_{y_i}) \cdot (L_{\text{foot}} - 2 \cdot e_{x_i})}$$

$$\sigma_v = \begin{pmatrix} 1.149 \\ 1.697 \\ 1.512 \\ 1.198 \\ 0.899 \\ 1.568 \\ 1.324 \\ 1.054 \\ 0.93 \\ 1.05 \end{pmatrix} \cdot \text{ksf}$$

Average Soil Bearing Pressure (AASHTO LRFD 11.6.3.2)

$$\gamma_{\text{bear}} := \begin{pmatrix} \phi_{\text{bear_str}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ext}} \\ \phi_{\text{bear_ser}} \\ \phi_{\text{bear_ser}} \\ \phi_{\text{bear_ser}} \end{pmatrix} = \begin{pmatrix} 0.45 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Resistance factor for bearing pressure for each load case

$$q_{\text{all}} := \gamma_{\text{bear}} \cdot \begin{pmatrix} q_{\text{n.str}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \\ q_{\text{n.ser}} \end{pmatrix} = \begin{pmatrix} 2.3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \\ 3 \end{pmatrix} \cdot \text{ksf}$$

Allowable factored bearing resistance, conservatively use the service nominal bearing resistance for extreme event analysis

$$\frac{\sigma_v}{q_{\text{all}}} = \begin{pmatrix} 0.499 \\ 0.566 \\ 0.504 \\ 0.399 \\ 0.3 \\ 0.523 \\ 0.441 \\ 0.351 \\ 0.31 \\ 0.35 \end{pmatrix}$$

Bearing pressure demand to capacity ratio for each load case

$$\text{Check}_{\text{bear_press}} := \text{if} \left(\max \left(\frac{\sigma_v}{q_{\text{all}}} \right) \leq 1.0, \text{"Bearing Capacity Adequate"}, \text{"Increase Size of Footing"} \right)$$

Check_{bear_press} = "Bearing Capacity Adequate"

Check Footing Overturning

$$M_{x,u} = \begin{pmatrix} 0 \\ 381.936 \\ 381.936 \\ 0 \\ 0 \\ 286.452 \\ 286.452 \\ 153.068 \\ 0 \\ 114.801 \end{pmatrix} \cdot \text{kip-ft}$$

$$M_{y,u} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 27.002 \\ 27.002 \\ 20.252 \\ 20.252 \\ 0 \\ 27.002 \\ 20.252 \end{pmatrix} \cdot \text{kip-ft}$$

Recall: Overturning Moments (longitudinal and transverse)

$$M_{x,u.resist} := P_u \cdot \frac{L_{foot}}{2}$$

$$M_{x,u.resist} = \begin{pmatrix} 1176.139 \\ 1170.469 \\ 863.014 \\ 1170.469 \\ 863.014 \\ 1170.469 \\ 863.014 \\ 894.644 \\ 894.644 \\ 894.644 \end{pmatrix} \cdot \text{kip-ft}$$

Longitudinal Resisting moment due to axial load

$$M_{y,u.resist} := P_u \cdot \frac{B_{foot}}{2}$$

$$M_{y,u.resist} = \begin{pmatrix} 588.069 \\ 585.235 \\ 431.507 \\ 585.235 \\ 431.507 \\ 585.235 \\ 431.507 \\ 447.322 \\ 447.322 \\ 447.322 \end{pmatrix} \cdot \text{kip-ft}$$

Transverse resisting moment due to axial load in footing

$$\frac{M_{x,u}}{M_{x,u.resist}} = \begin{pmatrix} 0 \\ 0.326 \\ 0.443 \\ 0 \\ 0 \\ 0.245 \\ 0.332 \\ 0.171 \\ 0 \\ 0.128 \end{pmatrix} \quad \frac{M_{y,u}}{M_{y,u.resist}} = \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0.046 \\ 0.063 \\ 0.035 \\ 0.047 \\ 0 \\ 0.06 \\ 0.045 \end{pmatrix} \quad \text{Demand to capacity of overturning forces}$$

$$\text{Check}_{over} := \text{if} \left(\max \left(\frac{M_{x,u}}{M_{x,u.resist}}, \frac{M_{y,u}}{M_{y,u.resist}} \right) \leq 1.0, \text{"Overturning of Footing OK"}, \text{"Overturning in Footing NG"} \right)$$

Check_{over} = "Overturning of Footing OK"

Check Footing Sliding

C_{fact} := 1.0

Cohesion Factor
 (1.0 for concrete cast against soil)

$$R_t := C_{fact} \cdot P_u \cdot \tan(\delta_{soil.conc}) = \begin{pmatrix} 71.705 \\ 71.36 \\ 52.615 \\ 71.36 \\ 52.615 \\ 71.36 \\ 52.615 \\ 54.543 \\ 54.543 \\ 54.543 \end{pmatrix} \cdot \text{kip}$$

Nominal sliding resistance between soil and foundation (AASHTO LRFD 10.6.3.4-2)

$$\phi_{slide} := \begin{pmatrix} \phi_{slide_str} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ext} \\ \phi_{slide_ser} \\ \phi_{slide_ser} \\ \phi_{slide_ser} \end{pmatrix} = \begin{pmatrix} 0.8 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Sliding Resistance Factors

$$R_{r_i} := \phi_{slide_i} \cdot R_{t_i} \quad R_r = \begin{pmatrix} 57.364 \\ 71.36 \\ 52.615 \\ 71.36 \\ 52.615 \\ 71.36 \\ 52.615 \\ 54.543 \\ 54.543 \\ 54.543 \end{pmatrix} \cdot \text{kip}$$

Factored sliding resistance between soil and foundation (AASHTO LRFD 10.6.3.4-1)
 (Conservatively ignore passive resistance of soil)

$$V_{u_i} := \sqrt{(V_{x,u_i})^2 + (V_{y,u_i})^2} \quad V_u = \begin{pmatrix} 0 \\ 14.156 \\ 14.156 \\ 1.167 \\ 1.167 \\ 10.653 \\ 10.653 \\ 14.156 \\ 1.167 \\ 10.653 \end{pmatrix} \cdot \text{kip}$$

Factored Horizontal force in footing

$$Check_{slide} := \text{if} \left(\max \left(\frac{V_u}{R_r} \right) \leq 1.0, \text{"Footing Adequate for Sliding"}, \text{"Footing NOT Adequate for Sliding"} \right)$$

Check_{slide} = "Footing Adequate for Sliding"

Note: Twisting of the foundation was not checked based on engineering judgement. The torsional load on the foundation is not great enough to overcome frictional resistance of the soil.

Structural Design of Footing

Bending Design Forces

$$M_{u, \text{long. footing}} := \sigma_{v_i} \cdot B_{\text{foot}} \cdot \left(\frac{L_{\text{foot}} - L_{\text{ped}}}{2} \right)^2$$

$$M_{u, \text{long. foot}} = \begin{pmatrix} 194.109 \\ 286.739 \\ 255.509 \\ 202.517 \\ 151.939 \\ 264.936 \\ 223.693 \\ 178.128 \\ 157.137 \\ 177.419 \end{pmatrix} \cdot \text{kip}\cdot\text{ft}$$

Longitudinal Bending moment at face of column for each load case

$$M_{u, \text{trans. footing}} := \sigma_{v_i} \cdot L_{\text{foot}} \cdot \left(\frac{B_{\text{foot}} - W_{\text{ped}}}{2} \right)^2$$

$$M_{u, \text{trans. foot}} = \begin{pmatrix} 57.429 \\ 84.834 \\ 75.594 \\ 59.916 \\ 44.952 \\ 78.383 \\ 66.181 \\ 52.701 \\ 46.49 \\ 52.491 \end{pmatrix} \cdot \text{kip}\cdot\text{ft}$$

Transverse Bending moment at face of column for each load case

Check Bending - Longitudinal Direction

$$N_{\text{bar. long. bot}} := \text{Ceil} \left(\frac{B_{\text{foot}} - 2.6\text{in} - D_{\text{bar. main}}}{S_{\text{bot. long}}}, 1 \right) + 1 = 8$$

Number of bars in the bottom mat in the longitudinal direction

$$N_{\text{bar. long. top}} := \text{Ceil} \left(\frac{B_{\text{foot}} - 2.6\text{in} - D_{\text{bar. main}}}{S_{\text{top. long}}}, 1 \right) + 1 = 8$$

Number of bars in the top mat in the longitudinal direction

$$A_{s, \text{long}} := \frac{D_{\text{bar. main}}^2 \cdot \pi}{4} \cdot N_{\text{bar. long. bot}} = 6.283 \cdot \text{in}^2$$

Total area of steel in the bottom mat in the longitudinal direction

$$A'_{s, \text{long}} := \frac{D_{\text{bar. sec}}^2 \cdot \pi}{4} \cdot N_{\text{bar. long. top}} = 1.571 \cdot \text{in}^2$$

Total area of steel in the top mat in the longitudinal direction

$$d_{\text{long}} := t_{\text{foot}} - \text{clr}_{\text{bot}} - \frac{D_{\text{bar. main}}}{2} = 14.5 \cdot \text{in}$$

Distance from tensile reinforcement to extreme compression fiber (Bottom)

$$d'_{\text{long}} := \text{clr}_{\text{side}} + \frac{D_{\text{bar. main}}}{2} = 3.5 \cdot \text{in}$$

Distance from compression reinforcement to extreme compression fiber (Top)

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \leq 4\text{ksi} \\ \max \left[0.85 - 0.05 \cdot \left(\frac{f'_c - 4\text{ksi}}{\text{ksi}} \right), 0.65 \right] & \text{otherwise} \end{cases} = 0.8$$

Stress Block N.A. Parameter (AASHTO LRFD 5.7.2.2)

$c := 2 \text{ in}$

Initial guess for distance from extreme compression fiber to N.A.

Note: This check conservatively ignores the compression reinforcing.

Given

$A_{s, \text{long}} \cdot f_y = 0.85 f_c \cdot \beta_1 \cdot c \cdot (B_{\text{foot}})$

T=C Assume "tension" steel yields and "Compression" Steel acts in compression (AASHTO LRFD 5.7.3.1.2-4)

$c := \text{Find}(c) \quad c = 1.155 \text{ in}$

Distance from extreme compression fiber to N.A.

$a_{\text{long}} := c \cdot \beta_1 = 0.924 \text{ in}$

Equivalent Stress block depth

$\epsilon_s := \frac{0.003}{c} \cdot (d_{\text{long}} - c) = 0.0347$

Strain in "Tension" steel at concrete crushing

$$\phi_{\text{str}} := \begin{cases} 0.9 & \text{if } \epsilon_s \geq 0.005 \\ \max \left[0.75, 0.90 - (0.005 - \epsilon_s) \cdot \left(\frac{0.90 - 0.75}{0.005 - 0.002} \right) \right] & \text{otherwise} \end{cases} = 0.9$$

Resistance Factor for Concrete in Bending for strength limit states (AASHTO LRFD 5.5.4.2)

$\phi_{\text{EE}} := 1.0$

Resistance factor for concrete in bending for extreme event limit states

$\phi_{\text{serv}} := 1.0$

Resistance factor for concrete in bending for service limit states

$$\phi_i := \begin{cases} \phi_{\text{str}} & \text{if } i \leq i_{\text{str}} \\ \phi_{\text{EE}} & \text{if } i > i_{\text{str}} \wedge i \leq i_{\text{EE}} \\ \phi_{\text{serv}} & \text{if } i > i_{\text{EE}} \wedge i \leq i_{\text{ser}} \end{cases}$$

$$\phi_i = \begin{pmatrix} 0.9 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Resistance factors

$$\phi M_{n, \text{long}} := \phi \cdot \left[0.85 \cdot f_c \cdot a_{\text{long}} \cdot \left(d_{\text{long}} - \frac{a_{\text{long}}}{2} \right) \cdot B_{\text{foot}} \right]$$

$$\phi M_{n, \text{long}} = \begin{pmatrix} 396.915 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \\ 441.017 \end{pmatrix} \cdot \text{kip} \cdot \text{ft}$$

Longitudinal Factored Bending Resistance (AASHTO LRFD 5.7.3.2.1)

$$\text{Check}_{\text{long.bending}} := \text{if} \left(\max \left(\frac{M_{u,\text{long.foot}}}{\phi M_{n,\text{long}}} \right) \leq 1.0, \text{"Adequate in Bending (Long. Forces)"}, \text{"NG in Long. Bending"} \right)$$

Check_{long.bending} = "Adequate in Bending (Long. Forces)"

$$\frac{M_{u,\text{long.foot}}}{\phi M_{n,\text{long}}} = (0.489 \ 0.65 \ 0.579 \ 0.459 \ 0.345 \ 0.601 \ 0.507 \ 0.404 \ 0.356 \ 0.402)$$

Check Minimum Reinforcement - Longitudinal Direction

$$M_{cr} := \frac{\gamma_1 \gamma_3 f_r t_{\text{foot}}^2 B_{\text{foot}}}{6} = 248.528 \cdot \text{kip}\cdot\text{ft}$$

Cracking Moment (AASHTO LRFD 5.6.3.3-1)

$$\text{Check}_{\text{min.bend.rein.long}} := \begin{cases} \text{"Min. Longitudinal Reinforcement OK"} & \text{if } \max \left(\frac{M_{cr}}{\phi M_{n,\text{long}}} \right) \leq 1.0 \\ \text{"Min. Longitudinal Reinforcement. OK"} & \text{if } \max \left[\frac{1.33 \cdot (M_{u,\text{long.foot}})}{(\phi M_{n,\text{long}})} \right] \leq 1.0 \\ \text{"Min. Longitudinal Reinforcement Not OK"} & \text{otherwise} \end{cases}$$

$$\max \left(\frac{M_{cr}}{\phi M_{n,\text{long}}} \right) = 0.626 \quad \max \left[\frac{1.33 \cdot (M_{u,\text{long.foot}})}{(\phi M_{n,\text{long}})} \right] = 0.865$$

Check_{min.bend.rein.long} = "Min. Longitudinal Reinforcement. OK"

Check minimum reinforcement per (AASHTO LRFD 5.7.3.3.2)

Check Crack Control - Longitudinal Direction

$$\rho_{\text{long}} := \frac{A_{s,\text{long}}}{d_{\text{long}} B_{\text{foot}}} = 0.0045$$

Reinforcement Ratio

$$k_{\text{long}} := \sqrt{2 \cdot \rho_{\text{long}} \cdot n + (\rho_{\text{long}} \cdot n)^2} - \rho_{\text{long}} \cdot n = 0.223$$

$$j_{\text{long}} := 1 - \frac{k_{\text{long}}}{3} = 0.926$$

$$M_{s,\text{long}} := \max(\text{submatrix}(M_{u,\text{long.foot}}, i_{EE} + 1, i_{ser}, 0, 0))$$

$$M_{s,\text{long}} = 178.128 \cdot \text{kip}\cdot\text{ft}$$

Applied maximum positive service moment in the longitudinal direction at the base of the shaft

$$f_{ss,\text{long}} := \frac{M_{s,\text{long}}}{A_{s,\text{long}} j_{\text{long}} d_{\text{long}}} = 25.351 \cdot \text{ksi}$$

Stress in rebar at service moment

Ref. MacGregor, J.G., 2009, "Reinforced Concrete: Mechanics and Design, 5th Edition"

$$\text{Check}_{f_{ss,\text{long}}} := \text{if}(f_{ss,\text{long}} \leq 0.6f_y, \text{"Tensile Stress in Longitudinal Reinf. OK"}, \text{"Tensile Stress in Longitudinal Reinf. Not OK"})$$

Check_{f_{ss, long}} = "Tensile Stress in Longitudinal Reinf. OK"

(AASHTO LRFD 5.6.7)

$$d_c := t_{foot} - d_{long} = 3.5\text{-in} \quad \text{Distance to CG of Steel}$$

$$\beta_{s,long} := 1 + \frac{d_c}{0.70 \cdot (t_{foot} - d_c)} = 1.345 \quad \text{Crack Parameter (AASHTO LRFD 5.6.7-2)}$$

$$s_{max,long} := \frac{700 \cdot \frac{\text{kip}}{\text{in}} \cdot \gamma_e}{\beta_{s,long} \cdot f_{ss,long}} - 2 \cdot d_c = 13.533\text{-in} \quad \text{Maximum spacing, Base of Shaft, Longitudinal (AASHTO LRFD 5.6.7-1)}$$

$$\text{Check}_{crack,long} := \begin{cases} \text{"Reinf. Spacing for Cracking OK (Longitudinal)"} & \text{if } s_{max,long} \geq s_{bot,long} \\ \text{"Reinf. Spacing for Cracking Not OK (Longitudinal)"} & \text{otherwise} \end{cases}$$

Check_{crack,long} = "Reinf. Spacing for Cracking OK (Longitudinal)"

Check One Way Shear - Longitudinal Direction

$$d_{v,long} := \max\left(d_{long} - \frac{a_{long}}{2}, 0.9 \cdot d_{long}, 0.72 \cdot t_{foot}\right) = 1.17\text{-ft} \quad \text{Effective Shear Depth (AASHTO LRFD 5.7.2.8)}$$

$$V_{u,long,foot_i} := \sigma_{v_i} \cdot B_{foot} \cdot \left(\frac{L_{foot} - L_{ped}}{2} - d_{v,long}\right) \quad V_{u,long,foot} = \begin{matrix} 48.977 \\ 72.349 \\ 64.469 \\ 51.098 \\ 38.336 \\ 66.847 \\ 56.441 \\ 44.944 \\ 39.648 \\ 44.766 \end{matrix} \text{ kip}$$

Longitudinal Shear force at d_v from face of column for each load case

$$\text{Check}_{long, shear} := \text{if}\left(3 \cdot d_{v,long} \geq \frac{L_{foot} - L_{ped}}{2}, \text{"Simplified Shear Procedure OK"}, \text{"General Procedure Needed"}\right)$$

Check_{long, shear} = "General Procedure Needed" (AASHTO LRFD 5.7.3.4.1)

$$\epsilon_{s_gp} := \frac{\frac{M_{u,long,foot}}{d_{v,long}} + V_x \cdot WLx}{E_s \cdot A_{s,long}} = \begin{matrix} 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \\ 0.001 \end{matrix} \quad \text{Net Longitudinal Tensile Strain (AASHTO 5.7.3.4.2)}$$

$$\beta_{long} := \frac{4.8}{(1 + 750 \cdot \epsilon_{s_gp})} =$$

- (2.757)
- 2.322
- 2.452
- 2.711
- 3.013
- 2.412
- 2.601
- 2.849
- 2.979
- (2.853)

Factor indicating ability of diagonally cracked concrete to transmit tension and shear (AASHTO LRFD 5.7.3.4.2-1)

$$\theta_{long} := (29 + 3500 \epsilon_{s_gp}) \text{deg} =$$

- (32.459)
- 33.98
- 33.467
- 32.597
- 31.767
- 33.622
- 32.945
- 32.197
- 31.852
- (32.185)

Angle of inclination of diagonal compressive stresses (AASHTO LRFD 5.7.3.4.2-2)

$$V_{c, long} := \min(0.0316 \cdot \beta_{long} \cdot \sqrt{f'_c \cdot \text{ksi}} \cdot B_{foot} \cdot d_{v, long}, 0.25 \cdot f'_c \cdot B_{foot} \cdot d_{v, long})$$

Nominal shear resistance (AASHTO LRFD 5.7.3.3-1 / 5.7.3.3-2)

$$\phi_{V_i} := \begin{cases} \phi_{V, str} & \text{if } i \leq i_{str} \\ \phi_{V, ser} & \text{if } i > i_{str} \wedge i \leq i_{ser} \end{cases} \quad \phi_{V_i} =$$

- (0.9)
- 1
- 1
- 1
- 1
- 1
- 1
- 1
- 1
- 1

Resistance factors for shear

$$\phi V_{c, long_j} := \phi_{V_i} \cdot V_{c, long} \quad \phi V_{c, long} =$$

- (199.002)
- 221.113
- 221.113
- 221.113
- 221.113
- 221.113
- 221.113
- 221.113
- 221.113
- 221.113

kip Factored shear resistance

$$\text{Check}_{\text{shear.long}} := \text{if} \left(\max \left(\frac{V_{u.\text{long.foot}}}{\phi V_{c.\text{long}}} \right) \leq 1.0, \text{"Longitudinal Footing Shear OK"}, \text{"Longitudinal Footing Shear Not OK"} \right)$$

Check_{shear.long} = "Longitudinal Footing Shear OK"

$$\max \left(\frac{V_{u.\text{long.foot}}}{\phi V_{c.\text{long}}} \right) = 0.327$$

Check Bending - Transverse Direction

$$N_{\text{bar.trans.bot}} := \text{Ceil} \left(\frac{L_{\text{foot}} - 2.6\text{in} - D_{\text{bar.main}}}{S_{\text{bot.trans}}}, 1 \right) + 1 = 16$$

Number of bars in the bottom mat in the transverse direction

$$N_{\text{bar.trans.top}} := \text{Ceil} \left(\frac{L_{\text{foot}} - 2.6\text{in} - D_{\text{bar.main}}}{S_{\text{top.trans}}}, 1 \right) + 1 = 16$$

Number of bars in the top mat in the transverse direction

$$A_{s.\text{trans}} := \frac{D_{\text{bar.sec}}^2 \cdot \pi}{4} \cdot N_{\text{bar.trans.bot}} = 3.142 \cdot \text{in}^2$$

Total area of steel in the bottom mat in the transverse direction

$$A'_{s.\text{trans}} := \frac{D_{\text{bar.sec}}^2 \cdot \pi}{4} \cdot N_{\text{bar.trans.top}} = 3.142 \cdot \text{in}^2$$

Total area of steel in the top mat in the transverse direction

$$d_{\text{trans}} := t_{\text{foot}} - \text{clr}_{\text{bot}} - D_{\text{bar.main}} - \frac{D_{\text{bar.sec}}}{2} = 13.75\text{in}$$

Distance from tensile reinforcement to extreme compression fiber

$$d'_{\text{trans}} := \text{clr}_{\text{side}} + D_{\text{bar.main}} - \frac{D_{\text{bar.sec}}}{2} = 3.75\text{in}$$

Distance from compression reinforcement to extreme compression fiber

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \leq 4\text{ksi} \\ \max \left[0.85 - 0.05 \cdot \left(\frac{f_c - 4\text{ksi}}{\text{ksi}} \right), 0.65 \right] & \text{otherwise} \end{cases} = 0.8$$

Stress Block N.A. Parameter (AASHTO LRFD 5.7.2.2)

$$c := 2\text{in}$$

Initial guess for distance from extreme compression fiber to N.A.

Note: This check conservatively ignores the compression reinforcing.

Given

$$A_{s.\text{trans}} \cdot f_y = 0.85 f_c \cdot \beta_1 \cdot c \cdot (L_{\text{foot}})$$

T=C Assume "tension" steel yields and "Compression" Steel acts in compression (AASHTO LRFD 5.7.3.1.2-4)

$$c := \text{Find}(c) \quad c = 0.289\text{in}$$

Distance from extreme compression fiber to N.A.

$$a_{\text{trans}} := c \cdot \beta_1 = 0.231\text{in}$$

Equivalent Stress block depth

$$\epsilon_s := \frac{0.003}{c} \cdot (d_{\text{trans}} - c) = 0.1399$$

Strain in "Tension" steel at concrete crushing

$$\phi_{str} := \begin{cases} 0.9 & \text{if } \epsilon_s \geq 0.005 \\ \max\left[0.75, 0.90 - (0.005 - \epsilon_s) \cdot \left(\frac{0.90 - 0.75}{0.005 - 0.002}\right)\right] & \text{otherwise} \end{cases} = 0.9$$

Resistance Factor for Concrete in Bending for strength limit states (AASHTO LRFD 5.5.4.2)

$\phi_{EE} := 1.0$

Resistance factor for concrete in bending for extreme event limit states

$\phi_{serv} := 1.0$

Resistance factor for concrete in bending for service limit states

$$\phi_i := \begin{cases} \phi_{str} & \text{if } i \leq i_{str} \\ \phi_{EE} & \text{if } i > i_{str} \wedge i \leq i_{EE} \\ \phi_{serv} & \text{if } i > i_{EE} \wedge i \leq i_{ser} \end{cases}$$

$$\phi_i = \begin{pmatrix} 0.9 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Resistance factors

$$\phi M_{n,trans} := \phi \left[0.85 \cdot f_c \cdot a_{trans} \cdot \left(d_{trans} - \frac{a_{trans}}{2} \right) \cdot L_{foot} \right]$$

$$\phi M_{n,trans} = \begin{pmatrix} 192.753 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \\ 214.17 \end{pmatrix} \cdot \text{kip}\cdot\text{ft}$$

Transverse Factored Bending Resistance (AASHTO LRFD 5.7.3.2.1)

$$\text{Check}_{trans.bending} := \text{if} \left(\max \left(\frac{M_{u,trans.foot}}{\phi M_{n,trans}} \right) \leq 1.0, \text{"Adequate in Bending (Trans. Forces)"}, \text{"NG in Trans. Bending"} \right)$$

Check_{trans.bending} = "Adequate in Bending (Trans. Forces)"

$$\frac{M_{u,trans.foot}}{\phi M_{n,trans}}^T = (0.298 \ 0.396 \ 0.353 \ 0.28 \ 0.21 \ 0.366 \ 0.309 \ 0.246 \ 0.217 \ 0.245)$$

Check Minimum Reinforcement - Transverse Direction

$$M_{cr} := \frac{\gamma_1 \cdot \gamma_3 \cdot f_r \cdot t_{foot}^2 \cdot L_{foot}}{6} = 497.055 \cdot \text{kip-ft}$$

Cracking Moment (AASHTO LRFD
5.7.3.3.2-1)

$$\max\left(\frac{M_{cr}}{\phi M_{n,trans}}\right) = 2.579$$

$$\max\left[\frac{1.33 \cdot (M_{u,trans,foot})}{(\phi M_{n,trans})}\right] = 0.527$$

$$\text{Check}_{\min.bend.rein.trans} := \begin{cases} \text{"Min. Transverse Reinforcement OK"} & \text{if } \max\left(\frac{M_{cr}}{\phi M_{n,trans}}\right) \leq 1.0 \\ \text{"Min. Transverse Reinforcement. OK"} & \text{if } \max\left[\frac{1.33 \cdot (M_{u,trans,foot})}{(\phi M_{n,trans})}\right] \leq 1.0 \\ \text{"Min. Transverse Reinforcement Not OK"} & \text{otherwise} \end{cases}$$

Check_{min.bend.rein.trans} = "Min. Transverse Reinforcement. OK"

Check minimum reinforcement
per (AASHTO LRFD 5.7.3.3.2)

Check Crack Control - Transverse Direction

$$\rho_{trans} := \frac{A_{s,trans}}{d_{trans} \cdot L_{foot}} = 0.0012$$

Reinforcement Ratio

$$k_{trans} := \sqrt{2 \cdot \rho_{trans} \cdot n + (\rho_{trans} \cdot n)^2} - \rho_{trans} \cdot n = 0.122$$

$$j_{trans} := 1 - \frac{k_{trans}}{3} = 0.959$$

$$M_{s,trans} := \max(\text{submatrix}(M_{u,trans,foot}, i_{EE} + 1, i_{ser}, 0, 0))$$

$$M_{s,trans} = 52.701 \cdot \text{kip-ft}$$

Applied maximum positive service moment in the
transverse direction at the base of the shaft

$$f_{ss,trans} := \frac{M_{s,trans}}{A_{s,trans} \cdot j_{trans} \cdot d_{trans}} = 15.261 \cdot \text{ksi}$$

Stress in rebar at service moment

Ref. MacGregor, J.G., 2009, "Reinforced Concrete:
Mechanics and Design, 5th Edition"

$$\text{Check}_{f_{ss,trans}} := \text{if}(f_{ss,trans} \leq 0.6f_y, \text{"Tensile Stress in Transverse Reinf. OK"}, \text{"Tensile Stress in Transverse Reinf. Not OK"})$$

Check_{f_{ss,trans}} = "Tensile Stress in Transverse Reinf. OK"

(AASHTO LRFD 5.6.7)

$$d_c := t_{foot} - d_{trans} = 4.25 \cdot \text{in}$$

Distance to CG of Steel

$$\beta_{s,trans} := 1 + \frac{d_c}{0.70 \cdot (t_{foot} - d_c)} = 1.442$$

Crack Parameter (AASHTO LRFD 5.6.7-2)

$$s_{\text{max.trans}} := \frac{700 \cdot \frac{\text{kip}}{\text{in}} \cdot \gamma_e}{\beta_{\text{s.trans}} \cdot f_{\text{ss.trans}}} - 2d_c = 23.319 \cdot \text{in}$$

Maximum spacing, Base of Shaft, Transverse
(AASHTO 5.6.7-1)

$$\text{Check}_{\text{crack.trans}} := \begin{cases} \text{"Reinf. Spacing for Cracking OK (Transverse)"} & \text{if } s_{\text{max.trans}} \geq S_{\text{bot.trans}} \\ \text{"Reinf. Spacing for Cracking Not OK (Transverse)"} & \text{otherwise} \end{cases}$$

Check_{crack.trans} = "Reinf. Spacing for Cracking OK (Transverse)"

Check One Way Shear - Transverse Direction

$$d_{\text{v.trans}} := \max\left(d_{\text{trans}} - \frac{a_{\text{trans}}}{2}, 0.9 \cdot d_{\text{trans}}, 0.72 \cdot t_{\text{foot}}\right) = 1.136 \cdot \text{ft}$$

Effective Shear Depth (AASHTO LRFD 5.8.2.9)

$$V_{\text{u.trans.foot}_i} := \sigma_{\text{v}_i} \cdot L_{\text{foot}} \left(\frac{B_{\text{foot}} - W_{\text{ped}}}{2} - d_{\text{v.trans}} \right)$$

$$V_{\text{u.trans.foot}} = \text{ kip}$$

- (25.063)
- (37.023)
- (32.99)
- (26.148)
- (19.618)
- (34.208)
- (28.882)
- (22.999)
- (20.289)
- (22.908)

Transverse Shear force at d_v from face of column for each load case

$$\text{Check}_{\text{trans.shear}} := \begin{cases} 3 \cdot d_{\text{v.trans}} \geq \frac{B_{\text{foot}} - W_{\text{ped}}}{2}, \text{"Simplified Shear Procedure OK"} & \text{"General Procedure Needed"} \end{cases}$$

Check_{trans.shear} = "Simplified Shear Procedure OK"

AASHTO LRFD 5.7.3.4.1

$$\beta_{\text{trans}} := 2.0$$

Factor indicating ability of diagonally cracked concrete to transmit tension and shear (AASHTO LRFD 5.7.3.4.1)

$$\theta_{\text{trans}} := 45 \text{deg}$$

Angle of inclination of diagonal compressive stresses (AASHTO LRFD 5.7.3.4.1)

$$V_{\text{c.trans}} := \min\left(0.0316 \cdot \beta_{\text{trans}} \cdot \sqrt{f_c} \cdot \text{ksi} \cdot L_{\text{foot}} \cdot d_{\text{v.trans}}, 0.25 \cdot f_c \cdot L_{\text{foot}} \cdot d_{\text{v.trans}}\right)$$

Nominal shear resistance (AASHTO LRFD 5.7.3.3-1 / 5.7.3.3-2)

$$\phi_{\text{v}_i} := \begin{cases} \phi_{\text{v.str}} & \text{if } i \leq i_{\text{str}} \\ \phi_{\text{v.ser}} & \text{if } i > i_{\text{str}} \wedge i \leq i_{\text{ser}} \end{cases} \quad \phi_{\text{v}_i} = \begin{pmatrix} 0.9 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Resistance factors for shear

$$\phi V_{c.trans_i} := \phi V_i V_{c.trans} \quad \phi V_{c.trans} = \begin{pmatrix} 332.955 \\ 369.95 \\ 369.95 \\ 369.95 \\ 369.95 \\ 369.95 \\ 369.95 \\ 369.95 \\ 369.95 \end{pmatrix} \cdot \text{kip} \quad \text{Factored shear resistance}$$

$$\text{Check}_{\text{shear.trans}} := \text{if} \left(\max \left(\frac{V_{u.trans.foot}}{\phi V_{c.trans}} \right) \leq 1.0, \text{"Transverse Footing Shear OK"}, \text{"Transverse Footing Shear Not OK"} \right)$$

Check_{shear.trans} = "Transverse Footing Shear OK"

$$\max \left(\frac{V_{u.trans.foot}}{\phi V_{c.trans}} \right) = 0.1$$

Check Two-Way Shear

$$V_{u.two} := P_u = \begin{pmatrix} 147.017 \\ 146.309 \\ 107.877 \\ 146.309 \\ 107.877 \\ 146.309 \\ 107.877 \\ 111.831 \\ 111.831 \\ 111.831 \end{pmatrix} \cdot \text{kip}$$

Shear force for two-way shear, conservatively assumed full axial load of the footing.

$$B_c := 1.0$$

Ratio of long to short side of rectangular column, (column is square)

$$b_o := \left[L_{ped} + \frac{(d_{v.long} + d_{v.trans})}{2} \right] \cdot 4 = 16.612 \text{ft}$$

Critical Section Perimeter (AASHTO LRFD 5.12.8.6.3)

$$V_{n.two} := \min \left[\left(0.063 + \frac{0.126}{B_c} \right) \cdot \sqrt{f_c \cdot \text{ksi}} \cdot b_o \cdot \frac{(d_{v.long} + d_{v.trans})}{2}, 0.126 \cdot \sqrt{f_c \cdot \text{ksi}} \cdot b_o \cdot \frac{(d_{v.long} + d_{v.trans})}{2} \right] = 777.104 \cdot \text{kip}$$

Nominal two-way shear resistance (AASHTO LRFD 5.12.8.6.3-2)

$$\phi V_{n,two_i} := \phi V_i \cdot V_{n,two} \quad \phi V_{n,two} = \begin{pmatrix} 699.394 \\ 777.104 \\ 777.104 \\ 777.104 \\ 777.104 \\ 777.104 \\ 777.104 \\ 777.104 \\ 777.104 \end{pmatrix} \text{ kip} \quad \text{Factored two-way shear resistance}$$

$$\text{Check}_{\text{shear_two}} := \text{if} \left(\max \left(\frac{V_{u,two}}{\phi V_{n,two}} \right) \leq 1.0, \text{"Footing Two-Way Shear OK"}, \text{"Footing Two-Way Shear Not OK"} \right)$$

Check_{shear_two} = "Footing Two-Way Shear OK"

Check Temperature and Shrinkage Steel

$$A_{s,\text{bar.main}} := \frac{D_{\text{bar.main}}^2 \cdot \pi}{4} = 0.785 \cdot \text{in}^2 \quad \text{Area of a single bar used main reinforcement}$$

$$A_{s,\text{bar.sec}} := \frac{D_{\text{bar.sec}}^2 \cdot \pi}{4} = 0.196 \cdot \text{in}^2 \quad \text{Area of a single bar used in secondary reinforcement}$$

$$A_{ts,\text{min}} := \min \left[\max \left[\frac{1.30 \cdot \frac{\min(B_{\text{foot}}, L_{\text{foot}})}{\text{in}} \cdot \frac{t_{\text{foot}}}{\text{in}}}{2 \cdot \left(\frac{\min(B_{\text{foot}}, L_{\text{foot}})}{\text{in}} + \frac{t_{\text{foot}}}{\text{in}} \right) \cdot \frac{f_y}{\text{ksi}}}, 0.11 \right], 0.60 \right] \cdot \frac{\text{in}^2}{\text{ft}} = 0.164 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum Temperature and Shrinkage Steel (AASHTO 5.10.8)}$$

$$S_{ts} := \min \left(\frac{A_{s,\text{bar.sec}}}{A_{ts,\text{min}}}, 12 \text{in} \right) = 1 \text{ft} \quad \text{Spacing of secondary bars used as temperature and shrinkage bars (AASHTO 5.10.8)}$$

$$A_{ts,\text{min}} := \frac{A_{s,\text{bar.sec}}}{S_{ts}} = 0.196 \cdot \frac{\text{in}^2}{\text{ft}} \quad \text{Minimum area of steel used for temperature and shrinkage steel}$$

$$\text{Check}_{ts_steel} := \begin{cases} \text{"Temp. and Shrinkage Reinforcement Not Adequate"} & \text{if } S_{ts} < \max(S_{\text{bot.long}}, S_{\text{top.long}}, S_{\text{bot.trans}}, S_{\text{top.trans}}) \\ \text{"Temp. and Shrinkage Reinforcement Not Adequate"} & \text{if } A_{ts,\text{min}} > \min \left(\frac{A_{s,\text{bar.main}}}{S_{\text{bot.long}}}, \frac{A_{s,\text{bar.sec}}}{S_{\text{top.long}}}, \frac{A_{s,\text{bar.sec}}}{S_{\text{bot.trans}}}, \frac{A_{s,\text{bar.sec}}}{S_{\text{top.trans}}} \right) \\ \text{"Temp. and Shrinkage Reinforcement Adequate"} & \text{otherwise} \end{cases}$$

Check_{ts_steel} = "Temp. and Shrinkage Reinforcement Adequate"

Pedestal Reinforcing

$$\text{Cover} := 3\text{in}$$

Pedestal reinforcement cover

$$N_{\text{bar.vert}} := 20 \quad 5 \text{ each face}$$

Total number of pedestal primary vertical reinforcing

$$D_{\text{bar.vert}} := \frac{8}{8}\text{in}$$

Pedestal primary vertical reinforcing size

$$S_{\text{bar.vert}} := \frac{36\text{in} - 2(3\text{in}) - 2(0.5\text{in}) - 1\text{in}}{5} = 5.6\text{in}$$

Pedestal primary vertical reinforcing spacing

$$A_{\text{s.bar.vert}} := \frac{D_{\text{bar.vert}}^2 \cdot \pi}{4 \cdot S_{\text{bar.vert}}} = 1.68 \frac{\text{in}^2}{\text{ft}}$$

Area of reinforcement of pedestal primary vertical reinforcing

$$D_{\text{bar.tie}} := \frac{5}{8}\text{in}$$

Pedestal horizontal tie reinforcing size

$$S_{\text{bar.tie}} := 12\text{in}$$

Pedestal horizontal tie spacing

$$A_{\text{s.bar.tie}} := \frac{D_{\text{bar.tie}}^2 \cdot \pi}{4 \cdot S_{\text{bar.tie}}} = 0.31 \frac{\text{in}^2}{\text{ft}}$$

Area of reinforcement of pedestal tie reinforcing

Pedestal Structural Design:

Assumptions:

- Only bending about the transverse axis is considered for the design of the pedestal.

Pedestal Slenderness (AASHTO 5.6.4.3)

$$I_{\text{ped.x}} := \frac{1}{12} \cdot W_{\text{ped}} \cdot L_{\text{ped}}^3 = 6.75\text{ft}^4$$

Pedestal moment of inertia about X-axis

$$I_{\text{ped.y}} := \frac{1}{12} \cdot L_{\text{ped}} \cdot W_{\text{ped}}^3 = 6.75\text{ft}^4$$

Pedestal moment of inertia about Y-axis

$$A_{\text{ped}} := W_{\text{ped}} \cdot L_{\text{ped}} = 9\text{ft}^2$$

Pedestal area

$$K_{\text{ped}} := 1$$

Effective length factor

$$r_{\text{ped.x}} := \sqrt{\frac{I_{\text{ped.x}}}{A_{\text{ped}}}} = 0.866\text{ft}$$

Radius of gyration about X-axis

ASHTO C5.6.4.3 Appr

$$r_{\text{ped.y}} := \sqrt{\frac{I_{\text{ped.y}}}{A_{\text{ped}}}} = 0.866\text{ft}$$

Radius of gyration about Y-axis

$$r_x := 0.3 \cdot L_{\text{ped}} = 0.9\text{ft}$$

$$r_z := 0.3 \cdot W_{\text{ped}} = 0.9\text{ft}$$

$$\frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.x}}} = 5.196$$

$$\frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.y}}} = 5.196$$

Column slenderness ratio about X-axis and Y-axis, respectively

$$\text{Check}_{\text{slenderness}} := \text{if} \left(\max \left(\frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.x}}}, \frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.y}}} \right) < 22, \text{"Neglect Slenderness Effects"}, \text{"Include Slenderness Effects"} \right)$$

Check_{slenderness} = "Neglect Slenderness Effects"

$$\text{DemCap}_{\text{slenderness}} := \frac{\max \left(\frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.x}}}, \frac{K_{\text{ped}} \cdot H_{\text{ped}}}{r_{\text{ped.y}}} \right)}{22} = 0.24$$

Limits of Reinforcing (AASHTO 5.6.4.2)

$$A_{s,\text{ped}} := N_{\text{bar.vert}} \cdot \frac{D_{\text{bar.vert}}^2 \cdot \pi}{4} = 15.708 \cdot \text{in}^2$$

Area of pedestal vertical (primary) reinforcement

$$A_{\text{ped}} = 1296 \cdot \text{in}^2$$

Area of pedestal

$$\frac{A_{s,\text{ped}}}{A_{\text{ped}}} = 0.012$$

Ratio of reinforcing steel area to gross concrete area

$$\text{Check}_{\text{max.reinf}} := \text{if} \left(\frac{A_{s,\text{ped}}}{A_{\text{ped}}} \leq 0.08, \text{"Max.Reinforcement OKAY"}, \text{"Max. Reinforcement NG"} \right)$$

Check_{max.reinf} = "Max.Reinforcement OKAY"

Maximum reinforcement (AASHTO LRFD 5.6.4.2-1)

$$\text{DemCap}_{\text{max.reinf}} := \frac{\left(\frac{A_{s,\text{ped}}}{A_{\text{ped}}} \right)}{0.08} = 0.15$$

$$\text{Check}_{\text{min.reinf.1}} := \text{if} \left(\frac{A_{s,\text{ped}}}{A_{\text{ped}}} \geq 0.135 \cdot \frac{f_c}{f_y}, \text{"Min.Reinforcement OKAY"}, \text{"Min. Reinforcement NG, Check 5.6.4.2-4"} \right)$$

Check_{min.reinf.1} = "Min.Reinforcement OKAY"

Minimum reinforcement (AASHTO LRFD 5.6.4.2-3)

Limit $0.135 \cdot \frac{f_c}{f_y} = 0.011$

$$\text{DemCap}_{\text{min.reinf.1}} := \frac{\left(0.135 \cdot \frac{f_c}{f_y} \right)}{\left(\frac{A_{s,\text{ped}}}{A_{\text{ped}}} \right)} = 0.93$$

Per AASHTO 5.6.4.2, if unfactored permanent design loads are less than $0.4 \cdot A_{\text{ped}} \cdot f_c$, the minimum reinforcing need not be greater than the following:

$$\max(P_U) = 147.017 \cdot \text{kip}$$

Maximum factored design load

$$0.4 \cdot A_{\text{ped}} \cdot f_c = 2592 \cdot \text{kip}$$

Limit of minimum reinforcing for 5.6.4.2-3

$$\text{Check}_{\text{min.reinf.2}} := \begin{cases} \text{"Min.Reinforcement OKAY"} & \text{if } \left[\left(\max(P_u) \leq 0.4 \cdot A_{\text{ped}} \cdot f_c \right) \wedge \frac{A_{s,\text{ped}}}{A_{\text{ped}}} \leq 0.015 \right] \\ \text{"Min. Reinforcement NG or Check Not Req'd"} & \text{otherwise} \end{cases}$$

Check_{min.reinf.2} = "Min.Reinforcement OKAY"

Minimum reinforcement
(AASHTO LRFD 5.6.4.2-4)

$$\text{DemCap}_{\text{min.reinf.2}} := \frac{\left(\frac{A_{s,\text{ped}}}{A_{\text{ped}}} \right)}{0.015} = 0.81$$

Axial Resistance (5.6.4.4)

$$\phi_c := 0.80$$

Compression strength resistance factor
(AASHTO 5.5.4.2)

$$k_c := 0.85$$

Ratio of maximum compressive strength to
design compressive strength of concrete

$$P_{n,\text{ped}} := 0.8 \cdot \left[k_c \cdot f_c \cdot (A_{\text{ped}} - A_{s,\text{ped}}) + f_y \cdot A_{s,\text{ped}} \right] = 5.107 \times 10^3 \cdot \text{kip}$$

Nominal compression resistance,
for members with tie reinforcement

$$\phi P_{n,\text{ped}} := \phi_c \cdot P_{n,\text{ped}} = 4085.58 \cdot \text{kip}$$

Design factored compression resistance,
for members with tie reinforcement

$$\text{Check}_{\text{ped.comp}} := \text{if} \left(\max(P_u) \leq \phi P_{n,\text{ped}}, \text{"Axial Compression OKAY"}, \text{"Axial Compression NG"} \right)$$

$$\max(P_u) = 147.017 \cdot \text{kip}$$

Check_{ped.comp} = "Axial Compression OKAY"

$$\text{DemCap}_{\text{ped.comp}} := \frac{\max(P_u)}{\phi P_{n,\text{ped}}} = 0.04$$

Transverse Flexural Resistance of Pedestal:

(AASHTO 5.6.3.2)

$$A_s := A_{s,\text{bar.vert}} = 1.683 \cdot \frac{\text{in}^2}{\text{ft}}$$

$$A'_s := A_{s,\text{bar.vert}} = 1.683 \cdot \frac{\text{in}^2}{\text{ft}}$$

$$d := W_{\text{ped}} - \text{Cover} - \frac{D_{\text{bar.vert}}}{2} - D_{\text{bar.tie}} = 31.875 \cdot \text{in}$$

$$d' := \text{Cover} + \frac{D_{\text{bar.vert}}}{2} + D_{\text{bar.tie}} = 4.125 \cdot \text{in}$$

$$\beta_1 := \begin{cases} 0.85 & \text{if } f_c \leq 4 \text{ksi} \\ \max \left[0.85 - 0.05 \cdot \left(\frac{f_c - 4 \text{ksi}}{\text{ksi}} \right), 0.65 \right] & \text{otherwise} \end{cases} \quad \beta_1 = 0.80$$

Stress Block to N.A. Parameter
[AASHTO 5.7.2.2]

Guess $c_0 := 2.0 \cdot \text{in}$

Guess at Neutral Axis

Given

T = C, Assuming 'Tension' steel yields
& 'Compression' steel is in tension also

$$A_s \cdot f_y = -A'_s \cdot \max \left[\min \left[E_s \cdot \left[\frac{0.003}{c_0} \cdot (d' - c_0) \right], f_y \right], -f_y \right] + 0.85 \cdot f_c \cdot \beta_1 \cdot c_0$$

$$c := \text{Find}(c_0) \quad c = 3.331 \cdot \text{in}$$

Distance from Extreme Comp. Fiber to N.A.

$$a := \beta_1 \cdot c \quad a = 2.665 \cdot \text{in}$$

Equivalent Stress Block Depth

$$\epsilon_s := \frac{0.003}{c} \cdot (d - c)$$

$$\epsilon_s = 0.0257$$

Strain in 'Tension' Steel at Concrete Crushing

$$F'_s := \max \left[\min \left[E_s \cdot \left[\frac{0.003}{c} \cdot (d - c) \right], f_y \right], -f_y \right]$$

$$F'_s = 20.746 \text{ ksi}$$

Stress in 'Compression' Reinforcement (Positive is Tension force)

$$F_s := \max \left[\min \left[E_s \cdot \left[\frac{0.003}{c} \cdot (d - c) \right], f_y \right], -f_y \right]$$

$$F_s = 60 \text{ ksi}$$

Stress in 'Tension' Reinforcement (Positive is Tension force)

$$M_W := A_s \cdot f_y \cdot \left(d - \frac{a}{2} \right) = 257.016 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Flexural resistance of pedestal

$$\phi_{\text{str}} := \begin{cases} 0.9 & \text{if } \epsilon_s > 0.005 \\ \max \left[0.75, 0.90 - (0.005 - \epsilon_s) \cdot \frac{0.90 - 0.75}{0.005 - 0.002} \right] & \text{otherwise} \end{cases} = 0.9$$

Resistance factor for concrete in bending for strength limit states (AASHTO 5.5.4.2)

$$\phi_{\text{ext}} := 1.0$$

Resistance factor for concrete in bending for extreme event limit states

$$\phi M_{\text{nx.str}} := \phi_{\text{str}} \cdot M_W = 231.315 \cdot \frac{\text{ft}}{\text{ft}}$$

Strength Factored Bending Resistance

$$\phi M_{\text{nx.ext}} := \phi_{\text{ext}} \cdot M_W = 257.016 \cdot \frac{\text{ft}}{\text{ft}}$$

Extreme Limit Factored Bending Resistance

$$M_{\text{u.x}} := \frac{\max(M_{\text{x.u}})}{L_{\text{ped}}} = 127.312 \cdot \frac{\text{ft}}{\text{ft}}$$

Maximum Factored Design Moment (Strength, Extreme Event, Service)

$$\text{Check}_{\text{str}} := \text{if} \left(\phi M_{\text{nx.str}} \geq M_{\text{u.x}}, \text{"Strength Capacity Adequate"} , \text{"Increase Reinforcement"} \right)$$

$$\text{Check}_{\text{str}} = \text{"Strength Capacity Adequate"}$$

$$\text{DemCap}_{\text{flex.str}} := \frac{M_{\text{u.x}}}{\phi M_{\text{nx.str}}} = 0.55$$

Note: Longitudinal reinforcement is checked considering torsion (per AASHTO 5.7.3.6.3) later in the calculations.

Check Development Length

$$L_{\text{dev.allow}} := t_{\text{foot}} - 3\text{in} = 15\text{in}$$

Allowable development length for the primary reinforcing

$$D_{\text{bar.vert}} = 1.0\text{in}$$

Size of primary reinforcing bar

$$L_{\text{dev.str}} := 25.76\text{in}$$

Primary reinforcing bar straight development length, per attached

$$L_{\text{dev.hook}} := 17\text{in}$$

Primary reinforcing bar hooked development length

$$\text{Check}_{\text{dev.long}} := \begin{cases} \text{"Straight Development Adequate"} & \text{if } L_{\text{dev.str}} \leq L_{\text{dev.allow}} \\ \text{"Hooked Development Needed"} & \text{if } L_{\text{dev.allow}} < L_{\text{dev.str}} \wedge L_{\text{dev.allow}} \geq L_{\text{dev.hook}} \\ \text{"Bar Can Not Be Fully Developed"} & \text{otherwise} \end{cases}$$

$$\text{Check}_{\text{dev.long}} = \text{"Bar Can Not Be Fully Developed"}$$

Note: The bar development shall be adequate due to strength demand is only approximate half of the capacity. Say OK

Minimum Reinforcement Check (AASHTO 5.6.3.3)

Because the section is monolithic and cast in place AASHTO Equation 5.6.3.3-1 has several terms that are not applicable (there is no prestressing stress and the composite and noncomposite section moduli are the same).

$$\gamma_1 := 1.6$$

Flexural Crack Variability Factor, CIP concrete

$$\gamma_3 := 0.67$$

Ratio of Yield to Rupture Strength, 60 ksi steel

$$M_{Cr} := \frac{\gamma_1 \cdot \gamma_3 \cdot L_{ped}^2}{6} \cdot f_r = 124.264 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

Design Cracking Moment

$$1.33 \cdot M_{u,x} = 169.325 \cdot \frac{\text{kip} \cdot \text{ft}}{\text{ft}}$$

$$\phi M_{nx.str} = 231.315 \cdot \frac{\text{ft}}{\text{ft}}$$

$$\text{Check}_{min.reinf} := \text{if}(\phi M_{nx.str} \geq \min(M_{Cr}, 1.33 \cdot M_{u,x}), \text{"Minimum reinforcement adequate"}, \text{"Add Reinforcement"})$$

$$\text{Check}_{min.reinf} = \text{"Minimum reinforcement adequate"}$$

$$\text{DemCap}_{min.reinf} := \frac{\min(M_{Cr}, 1.33 \cdot M_{u,x})}{\phi M_{nx.str}} = 0.5$$

Crack Control Check (AASHTO 5.6.7)

$$\gamma_e = 1$$

Exposure Factor - Class 1
(AASHTO LRFD 5.6.7)

$$d_c := \text{Cover} + \frac{D_{bar.vert}}{2} = 3.5 \cdot \text{in}$$

Concrete Cover

$$\rho_{rein} := \frac{A_s}{d} = 0.0044$$

Reinforcement Ratio

$$\beta_s := 1 + \frac{d_c}{0.7(t_{foot} - d_c)} = 1.345$$

Crack Depth Ratio

$$n := \left(\frac{E_s}{E_c} \right) = 7.126$$

Modular Ratio

$$k := \sqrt{(\rho_{rein} \cdot n)^2 + (2 \cdot \rho_{rein} \cdot n)} - \rho_{rein} \cdot n = 0.221$$

Working Stress Design

$$j := 1 - \frac{k}{3} = 0.926$$

$$f_{ss} := \frac{M_{u,x}}{A_s \cdot j \cdot d} = 30.743 \cdot \text{ks}$$

Service Tensile Stress(Reinforced
Concrete: Mechanics and Design)

$$\text{Check}_{ser.st} := \begin{cases} \text{"okay"} & \text{if } 0.6 \cdot f_y > f_{ss} \\ \text{"Add Steel"} & \text{otherwise} \end{cases}$$

Check Service Stress

$$\text{Check}_{ser.st} = \text{"okay"}$$

$$\text{DemCap}_{ser.st} := \frac{f_{ss}}{(0.6 \cdot f_y)} = 0.85$$

$$S_{min} := 700 \cdot \frac{\gamma_e}{\beta_s \cdot f_{ss}} \cdot \frac{kip}{in} - 2 \cdot d_c = 9.9 \cdot in$$

Min Reinf. Spacing for Cracks

$$Check_{sp} := \begin{cases} \text{"okay"} & \text{if } S_{bar.vert} \leq S_{min} \\ \text{"Reduce Spacing"} & \text{otherwise} \end{cases}$$

Check Minimum Spacing

$$Check_{sp} = \text{"okay"}$$

$$DemCap_{sp} := \frac{S_{bar.vert}}{S_{min}} = 0.56$$

Pedestal One-Way Shear (Flexural Shear)

per AASHTO BDS article 5.7.3.3

$$\phi_{V.str} := 0.9$$

Strength Resistance Factor

$$\phi_{V.ext} := 1.0$$

Extreme Event Resistance Factor

$$b_v := W_{ped} = 3 \text{ ft}$$

Effective Width (AASHTO 5.7.2.8)

$$d_v := \max\left(d - \frac{a}{2}, 0.9 \cdot d, 0.72 \cdot \min(L_{ped}, W_{ped})\right) = 30.543 \cdot in$$

Effective Depth (AASHTO 5.7.2.9)

$$\lambda := 1.0$$

Concrete Density Modification Factored (AASHTO 5.4.2.8)

$$A_{v.min} := .0316 \cdot \lambda \cdot \sqrt{f_c \cdot ksi} \cdot \frac{b_v \cdot S_{bar.tie}}{f_y} = 0.51 \cdot in^2$$

Minimum Required Transverse Reinforcing (AASHTO 5.7.2.5-1)

$$A_{v.prov} := 2 \cdot \frac{D_{bar.tie}^2 \cdot \pi}{4} = 0.614 \cdot in^2$$

Transverse Reinforcing Provided (2 legs per spacing)

$$Check_{shear.des} := \text{if}(A_{v.min} < A_{v.prov}, \text{"Simplified Procedure OKAY"}, \text{"General Procedure Needed"})$$

$$Check_{shear.des} = \text{"Simplified Procedure OKAY"}$$

Simplified Procedure

$$\beta_{str} := 2.0$$

Strength Limit Diagonal Crack Factor (AASHTO 5.7.3.4.1)

$$\beta_{ext} := 2.0$$

Extreme Limit Diagonal Crack Factor (AASHTO 5.7.3.4.1)

$$\theta_{str} := 45 \cdot \text{deg}$$

Strength Limit Crack Angle (AASHTO 5.7.3.4.1)

$$\theta_{ext} := 45 \cdot \text{deg}$$

Extreme Limit Crack Angle (AASHTO 5.7.3.4.1)

$$V_{c.str} := 0.0316 \cdot \beta_{str} \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi \cdot d_v = 51.795 \cdot \frac{kip}{ft}$$

$$0.25 \cdot f_c \cdot d_v = 458.14 \cdot \frac{kip}{ft}$$

Strength Limit Nominal Shear Resistance (AASHTO 5.7.3.3)

$$V_{n.str} := \min(V_{c.str}, 0.25 \cdot f_c \cdot d_v) = 51.795 \frac{\text{kip}}{\text{ft}}$$

$$V_{c.ext} := 0.0316 \cdot \beta_{ext} \sqrt{\frac{f_c}{\text{ksi}}} \cdot \text{ksi} \cdot d_v = 51.795 \frac{\text{kip}}{\text{ft}}$$

$$0.25 \cdot f_c \cdot d_v = 458.14 \frac{\text{kip}}{\text{ft}}$$

Extreme Limit Nominal Shear Resistance (AASHTO 5.7.3.3)

$$V_{n.ext} := \min(V_{c.ext}, 0.25 \cdot f_c \cdot d_v) = 51.795 \frac{\text{kip}}{\text{ft}}$$

$$\phi V_{n.str} := \frac{\phi_{v.str} \cdot V_{n.str}}{2} = 23.308 \frac{\text{kip}}{\text{ft}}$$

Strength Limit Shear Resistance (AASHTO 5.7.2-1)

$$\phi V_{n.ext} := \frac{\phi_{v.ext} \cdot V_{n.ext}}{2} = 25.898 \frac{\text{kip}}{\text{ft}}$$

Extreme Limit Shear Resistance (AASHTO 5.7.2-1)

$$V_{u.xz.ped} := \max\left(\frac{V_{x,u}}{L_{ped}}\right) = 4.719 \frac{\text{kip}}{\text{ft}}$$

Ultimate Factored Design Shear, use max X-axis or Z-axis value, capacity calculations compute the minimum capacity of either axis

$$\text{Check}_{\text{shear.str}} := \begin{cases} \text{"okay"} & \text{if } V_{u.xz.ped} < \phi V_{n.str} \\ \text{"Add Reinforcing"} & \text{otherwise} \end{cases}$$

Strength Limit Shear Capacity Check, no transverse reinforcement: (AASHTO 5.7.2.3-1)

$$\text{Check}_{\text{shear.str}} = \text{"okay"}$$

$$\text{DemCap}_{\text{shear.str}} := \frac{V_{u.xz.ped}}{\phi V_{n.str}} = 0.20$$

$$\text{Check}_{\text{tr.reif}} := \begin{cases} \text{"Okay, no transverse reinforcing req'd"} & \text{if } V_{u.xz.ped} < 0.5 \phi V_{n.str} \\ \text{"Add Transverse Reinforcing"} & \text{otherwise} \end{cases}$$

Transverse Reinforcing Check (AASHTO 5.7.2.3)

$$\text{Check}_{\text{tr.reif}} = \text{"Okay, no transverse reinforcing req'd"}$$

$$\text{DemCap}_{\text{tr.feinf}} := \frac{V_{u.xz.ped}}{0.5 \phi V_{n.str}} = 0.40$$

$$\text{Check}_{\text{min.tr.rein}} := \begin{cases} \text{"Transverse Reinforcing OK"} & \text{if } 2 \frac{(D_{\text{bar.tie}})^2 \cdot \pi}{4} \geq 0.0316 \cdot \lambda \cdot \sqrt{f_c \cdot \text{ksi}} \cdot \frac{W_{\text{ped}} \cdot S_{\text{bar.tie}}}{f_y} \\ \text{"Not Applicable"} & \text{if } \text{Check}_{\text{tr.reif}} = \text{"Okay, no transverse reinforcing req'd"} \\ \text{"Add Transverse Reinforcing"} & \text{otherwise} \end{cases}$$

Minimum Transverse Reinforcing (AASHTO 5.7.2.5)

$$\text{Check}_{\text{min.tr.rein}} = \text{"Not Applicable"}$$

$$\text{Check}_{\text{max.sp}} := \begin{cases} \text{"Transverse Reinforcing Spacing Okay"} & \text{if } S_{\text{bar.tie}} \leq \min(0.4 \cdot d_v, 12 \cdot \text{in}) \\ \text{"Decrease Transverse Reinforcing Sp"} & \text{otherwise} \end{cases}$$

Transverse Reinforcing Spacing Check (AASHTO 5.7.2.6)

$$\text{Check}_{\text{max.sp}} = \text{"Transverse Reinforcing Spacing Okay"}$$

$$\text{DemCap}_{\text{tr.feinf}} := \frac{S_{\text{bar.tie}}}{\min(0.4 \cdot d_v, 12 \cdot \text{in})} = 1.00$$

Interface Shear Strength

AASHTO 5.7.4

Area of Concrete in Transfer

$$A_{cv} := L_{ped} \cdot W_{ped} = 9 \cdot ft^2$$

Area of Reinforcing Across Interface:

$$A_{vf} := N_{bar.vert} \cdot \left(\frac{D_{bar.vert}^2 \cdot \pi}{4} \right) = 15.708 \cdot in^2$$

Assume concrete is placed against a clean surface that is not intentionally roughened (ignore shear key).

$$c := 0.075 \text{ ksi}$$

cohesion factor specified in Article 5.7.4.4

$$\mu := 0.6$$

friction factor specified in Article 5.7.4.4

$$K_1 := 0.2$$

fraction of concrete strength available to resist interface shear, as specified in Article 5.7.4.4

$$K_2 := 0.8 \text{ ksi}$$

limiting interface shear resistance specified in Article 5.7.4.4 (ksi)

$$V_{ni} := \min \left[c \cdot A_{cv} + \mu \cdot (A_{vf} \cdot f_y), K_1 \cdot f_c \cdot A_{cv}, K_2 \cdot A_{cv} \right] = 662.687 \cdot \text{kip}$$

AASHTO Eq. 5.7.4.3-3 (conservatively ignore permanent compression)

$$\phi_{int.shear} := \begin{pmatrix} 1.0 \\ 0.9 \\ 0.9 \end{pmatrix} \quad i_{load.case} := \begin{pmatrix} \text{"Extreme I"} \\ \text{"Strength I"} \\ \text{"Service I"} \end{pmatrix}$$

AASHTO 5.5.4.2, use resistance factors for shear per AASHTO 5.7.4.3. Extreme Event may be taken as 1.0 per AASHTO 5.7.4.3.

$$i := 0..2$$

Index counter

$$V_{r,i} := \left(\phi_{int.shear} \cdot V_{ni} \right) = \begin{pmatrix} 662.687 \\ 596.418 \\ 596.418 \end{pmatrix} \cdot \text{kip}$$

Reduced Interface Shear Resistance (AASHTO 5.7.4.3-1)

$$\text{Interface_Shear}_{0,i} := \begin{cases} \text{"Good"} & \text{if } (V_{x,u}^T)_{0,i} < (V_{r,i}^T)_{0,i} \\ \text{"Add Reinforcing"} & \text{otherwise} \end{cases}$$

Interface Shear Check (AASHTO 5.7.4.3)

$$\text{Interface_Shear} = (\text{"Good"} \text{ "Good"} \text{ "Good"})$$

$$\text{DemCap}_{\text{Interface_shear}} := \frac{\max(V_{x,u})}{\min(V_{r,i})} = 0.02$$

Interface Shear Minimum Reinforcing

$$\text{Interface_Reinforcing} := \begin{cases} \text{"Good"} & \text{if } A_{vf} \geq \frac{0.05 \cdot A_{cv} \cdot \text{ksi}}{f_y} \\ \text{"Add Reinforcing"} & \text{otherwise} \end{cases}$$

Check Minimum Interface Reinforcing (AASHTO 5.7.4.2)

$$\text{Interface_Reinforcing} = \text{"Good"}$$

$$\text{DemCap}_{\text{Interface_shear}} := \frac{0.05 \cdot A_{cv} \cdot \text{ksi}}{A_{vf} \cdot f_y} = 0.07$$

Torsion Check (AASHTO 5.7.2.1)

$$A_{cp} := L_{ped} \cdot W_{ped} = 9 \cdot \text{ft}^2$$

Area enclosed by outside perimeter of concrete cross section

$$K_c := 1.0$$

AASHTO 5.7.2.1-6 (for sections with no prestressing)

$$P_c := 2L_{ped} + 2 \cdot W_{ped} = 12 \text{ft}$$

Length of outside perimeter of the concrete section

$$\lambda := 1.0$$

Concrete density modification factor (AASHTO 5.4.2.8, for normal weight concrete)

$$\phi_{tor} := 0.9$$

Resistance factor for torsion in reinforced concrete sections (AASHTO 5.5.4.2)

$$T_{cr} := 0.126 \cdot K_c \cdot \lambda \cdot \sqrt{f'_c \cdot \text{ksi}} \cdot \frac{A_{cp}^2}{P_c} = 273.856 \cdot \text{ft} \cdot \text{kip}$$

Torsional cracking moment (AASHTO 5.7.2.1-4)

$$M_{z,u} := 64 \text{kip} \cdot \text{ft}$$

Torsion Demand

$$T_u := \max(M_{z,u}) = 64 \cdot \text{ft} \cdot \text{kip}$$

Recall factored torsional demand

$$\text{check}_{tor,1} := \text{if}(T_u \geq 0.25 \cdot \phi_{tor} \cdot T_{cr}, \text{"Consider Torsion"}, \text{"Neglect Torsion"})$$

AASHTO 5.7.2.1-3

$$\text{check}_{tor,1} = \text{"Consider Torsion"}$$

Closed stirrups are provided around the anchor rods to resist torsion.

$$A_o := \left[L_{ped} - 2 \cdot \left(\text{Cover} + \frac{D_{bar,tie}}{2} \right) \right] \cdot \left[W_{ped} - 2 \cdot \left(\text{Cover} + \frac{D_{bar,tie}}{2} \right) \right] = 5.992 \cdot \text{ft}^2$$

Area enclosed by the shear flow path (assume equal to area enclosed by stirrups)

$$A_t := \frac{D_{bar,tie}^2 \cdot \pi}{4} = 0.307 \cdot \text{in}^2$$

Area of one leg of closed transverse torsion reinforcement

$$s := S_{bar,tie} = 12 \cdot \text{in}$$

Spacing of torsion reinforcement

$$\theta_{ext} = 45 \cdot \text{deg}$$

Recall extreme limit crack angle

$$T_n := \frac{2 \cdot A_o \cdot A_t \cdot f_y}{s \cdot \tan(\theta_{ext})} = 220.61 \cdot \text{ft} \cdot \text{kip}$$

Nominal torsional resistance of section (AASHTO 5.7.3.6.2-1)

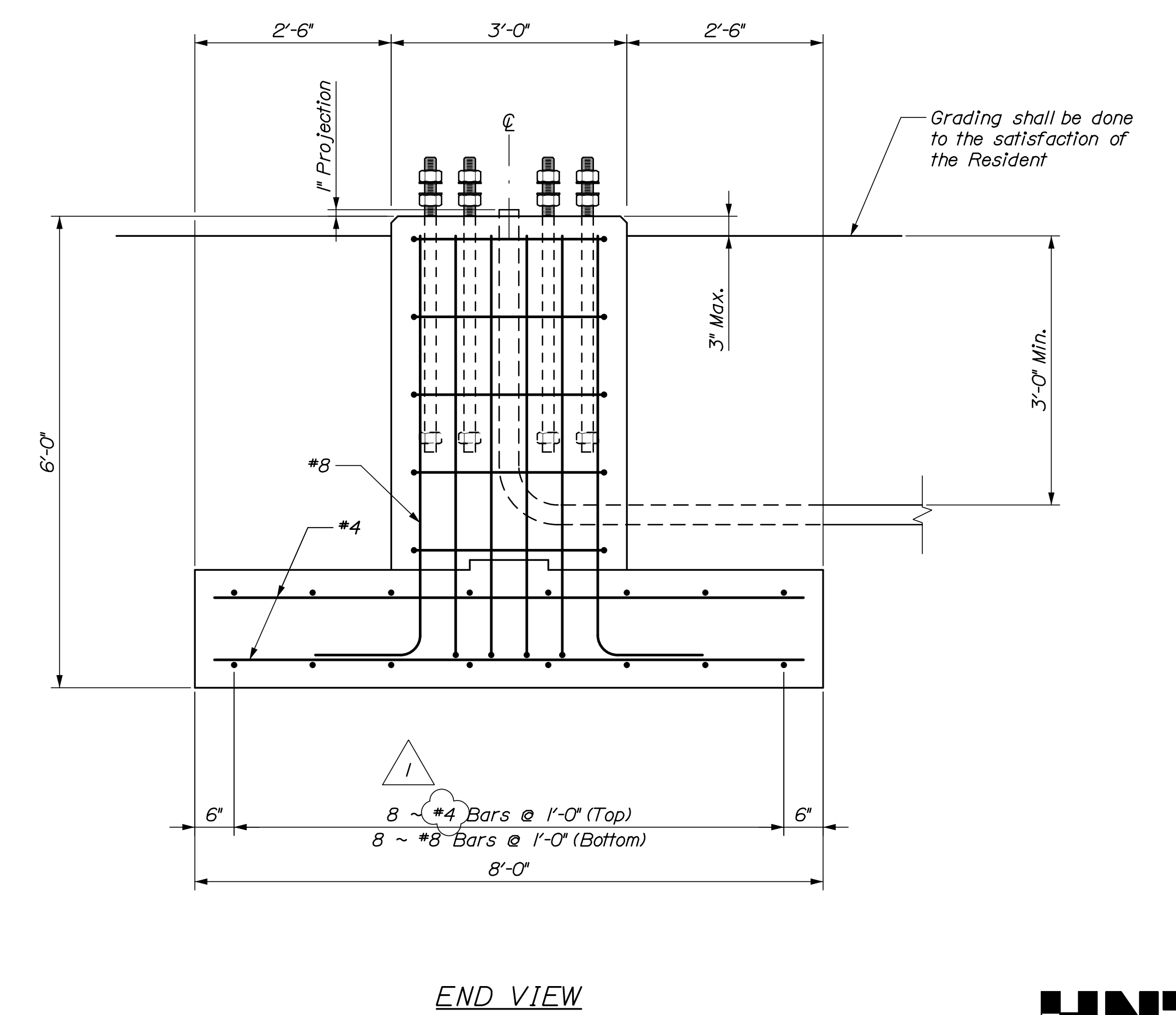
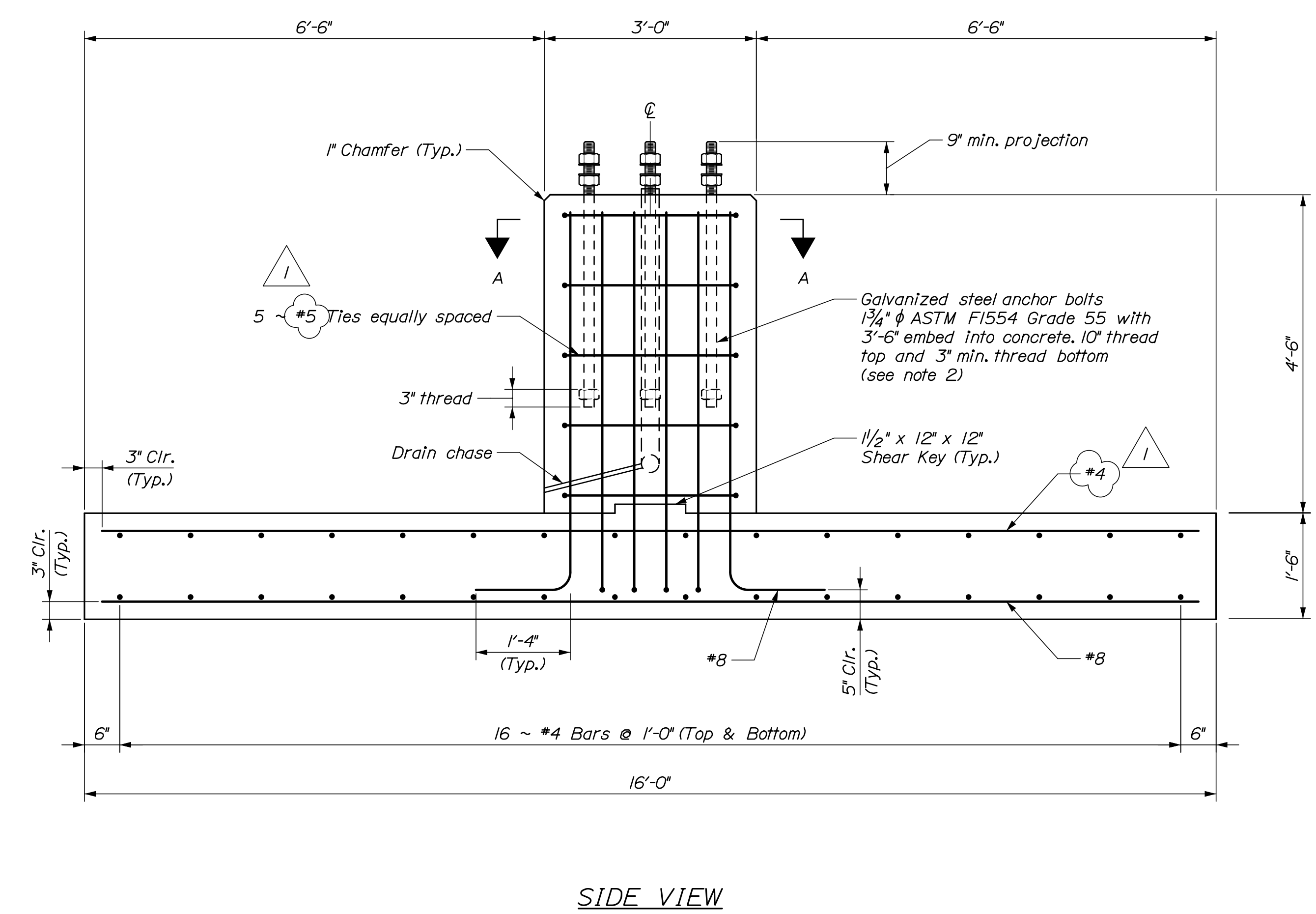
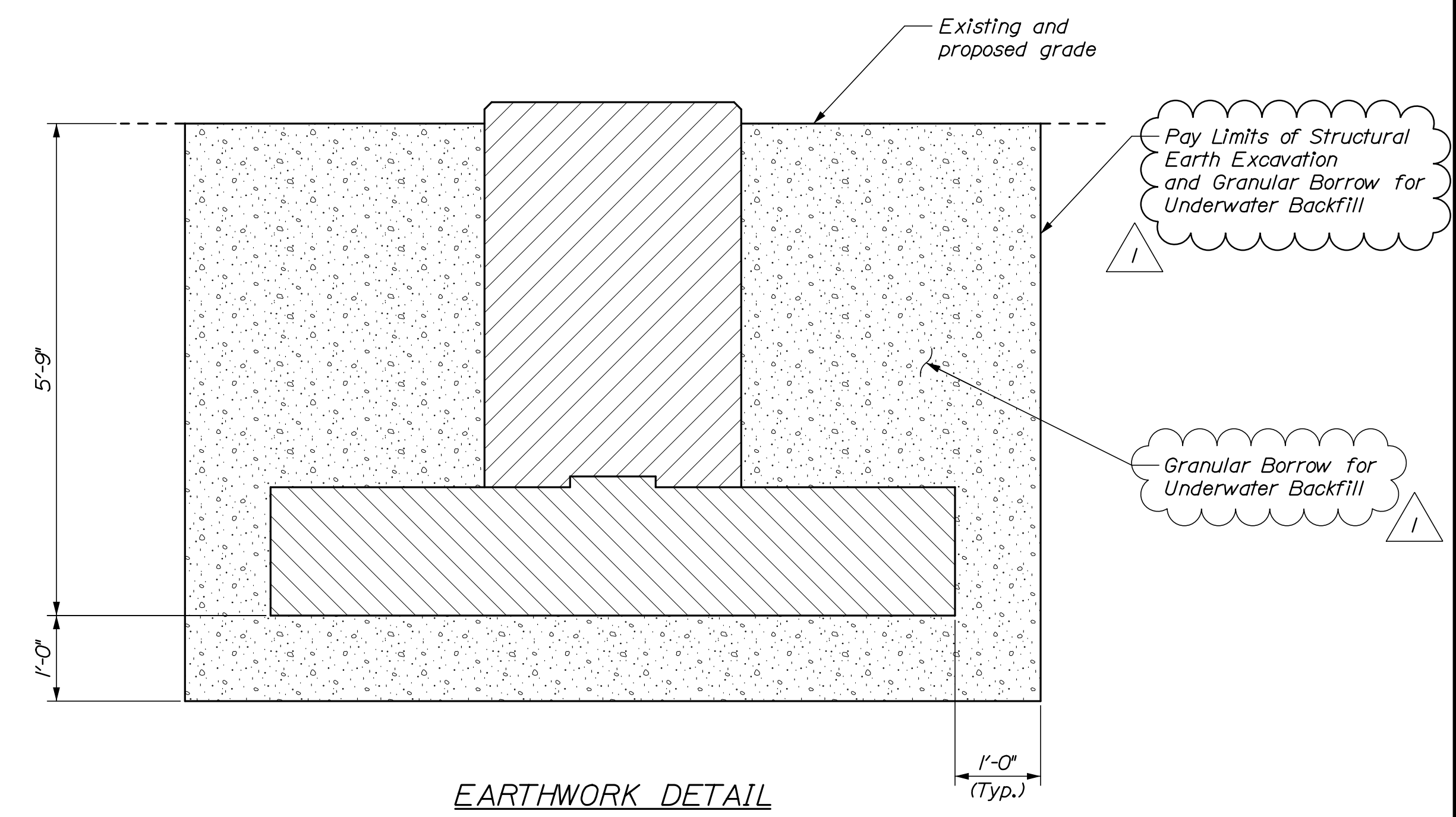
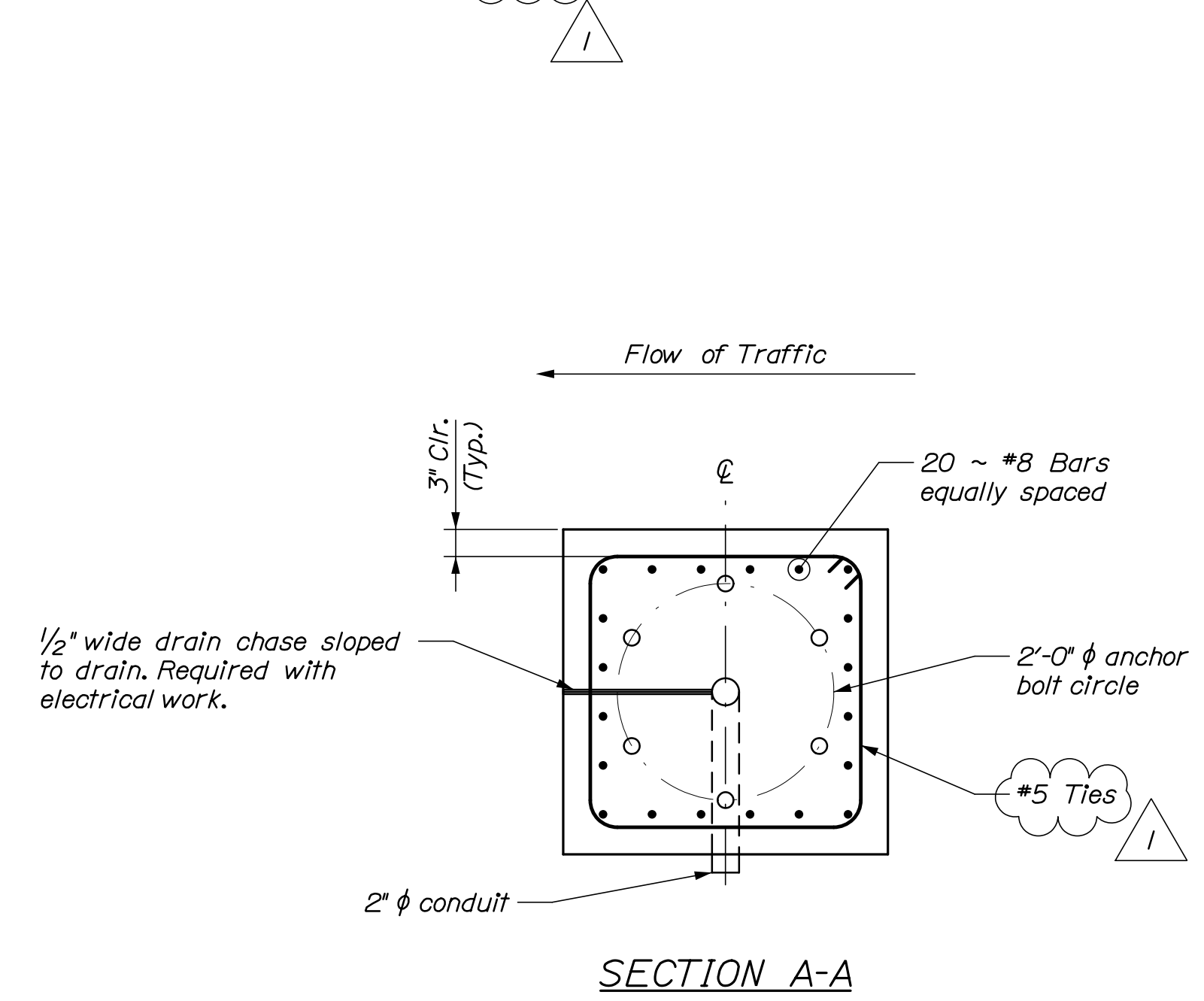
$$\text{check}_{tor,2} := \text{if}(\phi_{tor} \cdot T_n \geq T_u, \text{"OK"}, \text{"Add torsion reinforcement"})$$

$$\text{DemCap}_{torsion} := \frac{T_u}{\phi_{tor} \cdot T_n} = 0.322$$

$$\text{check}_{tor,2} = \text{"OK"}$$

FOUNDATION NOTES

1. All reinforcing steel shall be uncoated and comply with ASTM 615, Grade 60.
2. Steel anchor bolts and hardware shall be galvanized in accordance with ASTM A153.
3. A qualified geotechnical consultant shall be present to monitor materials used during construction and confirm that soil conditions and construction methods are consistent with the Schonewald Engineering Associates Geotechnical Engineering report of November 2022.



Date: 1/6/2023

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Filename: 006_specsignfundetails.dgn

STATE OF MAINE		DEPARTMENT OF TRANSPORTATION		2423900		WIN 024239.00	
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