



## **Nobles Wind Energy Facility**

### ***Wind Turbine Foundation Evaluation Report Repowering with a GE 1.6-91 and GE 1.6-97***

Prepared for  
Northern States Power Company  
d/b/a Xcel Energy Company

March 2021

DRAFT

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## Certifications

I hereby certify that this report was prepared by me or under my direct supervision and that I am a duly licensed Professional Engineer under the laws of the State of Minnesota.

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**ISSUED DRAFT FOR REVIEW**

Brian Tri, P.E.  
PE #: 48304

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March 31, 2021

Date

In addition to the Engineer of Record, the professional staff involved in the preparation of this report includes the following:

- Peter Blake, P.E., Barr Engineering Co.
- Eric Brandner, P.E., Barr Engineering Co.
- Jed Greenwood, P.E., P.G., Barr Engineering Co.
- Rob Osburn, P.E., Barr Engineering Co.
- Wes Karras, P.E., Barr Engineering Co.
- Matt Walker, P.E., Barr Engineering Co.
- Seth Strelow, Barr Engineering Co.
- Garrett Bayrd, Barr Engineering Co.

# 1.0 Executive Summary

Under authorization from Northern States Power Company, d/b/a Xcel Energy Company (Xcel), Barr Engineering Company (Barr) performed a structural assessment of the existing foundations at the Nobles Wind Energy Facility in Minnesota. Xcel proposes to replace the existing turbines with new turbines. The objectives of the structural assessment are as follows:

- Determine if the existing foundation could accommodate the design loads provided by GE for the GE1.6-91 and GE1.6-97 wind turbine using 2021 industry design standards or variations as further specified,
- Identify those foundation elements that do not meet standards, if any, and
- Make recommendations as to the course of action to be undertaken, if necessary.

The foundations passed all design checks. One item of note is that industry design practices for the anchorage of the tower to the foundation are different today than they were when the original designs were completed. Current design practice is to include vertical reinforcing steel bars surrounding the embedment ring and anchor bolt assembly to resist the entire pullout force exerted on the foundation. In the early and mid-2000's the anchorage was often designed to rely on the strength of a presumed shear failure surface within the concrete. This type of anchorage is generally allowed by **Standard 1**, although specific design measures are not provided and an explicit method to evaluate the fatigue resistance of this connection is not prescribed by any design code. For this evaluation, Barr developed a calculation procedure based on a design fatigue strength for this type of anchorage based on an analogous design formula and Barr's knowledge of historical connection performance. This alternative assessment procedure determined that the ultimate and fatigue pull-out strength met design requirements.

To validate the structural assessment, Barr performed instrumentation, monitoring, and visual condition inspection within a sample size of the existing foundations to verify that the pedestal, foundation, and connection are in good condition.

The *Wind Turbine Foundation Visual and Coring Inspection Summary Report* is included in **Appendix B**. The report summarizes the evaluation of 28 wind turbine foundations via the retrieval of cores extending from the surface of the footing towards the embedment plate under the pedestal. Some level of potential shear cracking or regions of poor consolidation were observed in cores within the following foundations:

- 16 (both cores),
- 39 (one core), and
- 47 (one core).

This type of cracking could be expected from the development of a pullout cone failure mechanism if the cracking plane crosses the axis of the core. No cracks were observed in the cores obtained from within the other 25 foundations.

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A *Foundation Structural Health Monitoring Summary Report* is also included in **Appendix C**. The monitoring efforts identified the following:

- 13 of the 14 foundations monitored (5, 6, 13, 25, 35, 40, 47, 58, 75, 77, 84, 99, and 104) appear to be functioning above the GE 1.6-91 repower turbine minimum rotational stiffness value of 30 GN-m/rad.
- All but two foundations monitored (6 and 73) appear to be functioning above the GE 1.6-97 repower turbine minimum rotational stiffness value of 50 GN-m/rad.

Barr's opinion is that the foundations in their current configuration are adequate to support the new GE turbines with 20 years of fatigue cycles/time given the following recommendations:

- (1) Perform structural health monitoring within 1 year on the three foundations (16, 39 and 47) with some level of cracking or poor consolidation identified within the cores.
- (2) Perform structural health monitoring again in 1 year on the two foundations (6 and 73) with measured average rotational stiffness values below 50 GN-m/rad.

The assessment described herein is based on the complete load documents provided by GE. Barr's analysis and calculations rely on the foundation loading provided by GE and are dependent on the load data being correct. The load data cannot be independently confirmed. The actual loads experienced by the foundation will likely vary depending on weather conditions and equipment performance.

## 2.0 Introduction

Xcel is considering the repowering of the existing wind turbines at the Nobles Facility. **Table 2-1** below lists the project name, project location, original foundation designer, the existing wind turbine, and the repower wind turbine model. The existing foundation design was based on a fatigue spectrum with a 20 year design life and Xcel would like to utilize the existing foundations for an additional 20 year life following the repowering.

**Table 2-1 Project, Location, Original Designer, Original Turbine, and Repower Turbine**

Project Name	State	Original Foundation Drawing IFC Date	Estimated Repowering Date	Original Foundation Designer	Existing Turbine	Repower Turbine
Nobles	Minnesota	December 2009	January 2021	Barr Engineering Co.	GE 1.5SLE-1.5MW 80m HH	GE 1.6-97 80m HH and GE 1.6-91 80m HH

Foundation design drawings and assessment calculations for the Nobles Wind Energy Facility are included in **Appendix A**.



## 3.0 References, Codes and Standards

### 3.1 Project Documents

Several project related documents were used to help with structural assessment of the foundations. These documents include Barr design drawing and GE foundation load document. The original geotechnical report is not referenced in this report but can be found on the Barr design drawing.

### 3.2 Codes, Standards, and Technical References

A detailed list of key codes, standards, and technical references is provided in **Section 8.2** of this report. These documents were used as a basis for this assessment and as a whole are called the “2021 standards”. A visual summary of these documents is shown on **Figure 3-1** below.

<p>“Overarching” industry document</p> <p>Industry codes and standards</p> <p>Technical References (specific components)</p>	ASCE/AWEA RP2011, Recommended Practice for Compliance of Large Land-Based Wind Turbine Support Structures, December 2011.			
	GL Wind Guidelines, Edition 2010.	IEC, Wind Turbine Generator Systems – Part 1: Safety Requirements, 3rd edition, 2005.	ICC Council Inc., International Building Code, 2018.	DNV-OS-C502, Offshore Concrete Structures, February 2018.
	<ol style="list-style-type: none"> <li>1) ACI 318-14, Building Code Requirements for Structural Concrete, 2014.</li> <li>2) ACI 318-19, Building Code Requirements for Structural Concrete (footing shear provisions), 2019.</li> <li>3) ACI 336R.4-XX, Guide/Analysis of Footings by Strength Design Method”, Draft 2008.</li> <li>4) AISC, Steel Construction Manual, 2016.</li> <li>5) Arya, S. O’Neill, M., Pincus, G., Structures/Foundations for Vibrating Machines, 1984.</li> <li>6) ASCE 7-16, Minimum Design Loads for Buildings and Other Structures, 2016.</li> <li>7) DNV, Riso National Lab., Guidelines for Design of Wind Turbines, 2nd Edition, 2002.</li> <li>8) Fahey M., “Soil stiffness values for foundation settlement analysis”, 1999.</li> <li>9) Potyondy, J.G., “Skin Friction between Soils and Concrete”, Geotechnique, Dec. 1961.</li> <li>10) Precast/Prestressed Concrete Institute, PCI Design Handbook, 5th Edition, 1999.</li> </ol>			

**Figure 3-1 Organization of Codes, Standards, and Technical References**

Wind turbines are somewhat unique structures with specialized concerns of highly eccentric loads and dynamic loading induced from rotating machinery. Often, structural design codes are typically intended for buildings or bridges and designers must be aware of areas where structural codes are not adequate. The result of this fact is that towers and foundations are designed using a combination of building codes and other standards:

- ASCE/AWEA joint committee produced a recommended practice document for design of wind energy structures that provides an overall framework for the design of wind turbine support structures, directing the designer to other design codes for specific elements of structural design.
- The GL Wind Guidelines provide guidance on the load cases to be considered and general guidance on design criteria for wind turbine support structures.
- IEC 61400 and ASCE 7 provide the basis for load development.

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- International Building Code (and incorporated ACI documents) is utilized for reinforced concrete design capacities, but does not address fatigue strength of reinforced concrete.
  - DNV's standard for offshore concrete structures is used for fatigue strength of reinforced concrete members.
  - Specific technical standards are incorporated for other design considerations.

**Section 8.2.1** lists documents referenced by building codes in the United States for the structural design of reinforced concrete foundations. **Section 8.2.2** lists industry standards which, while not referenced by building codes, were written specifically to apply to the design of wind turbine support structures. **Section 8.2.3** lists technical references that prescribe methods or approaches which were used in Barr's assessment.

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## 4.0 Design Loads

### 4.1 Wind Turbine Loads

#### 4.1.1 GE 1.6-91 80m HH and GE 1.6-97 80m HH

Wind turbine loads were calculated by GE in accordance with IEC 61400-1 (**Standard 4**) and presented in the foundation load documents (**Project Documents 1** and **2**).

#### 4.1.2 Wind Turbine Accidents and Abnormal Operation

Barr's analysis and calculations rely on the foundation loading provided by GE and are dependent on the GE load data being correct. The GE load data cannot be independently confirmed. The actual loads experienced by the foundation will likely vary depending on weather conditions and equipment performance.

### 4.2 Seismic Loads

The seismicity for the project site is low with the USGS seismic design maps indicating the unadjusted  $S_s$  and  $S_1$  values are 0.069 and 0.033 respectively (approximate location of the Nobles Wind Project). The seismic plus operating load is not a critical design load case.

### 4.3 Buoyancy Loads

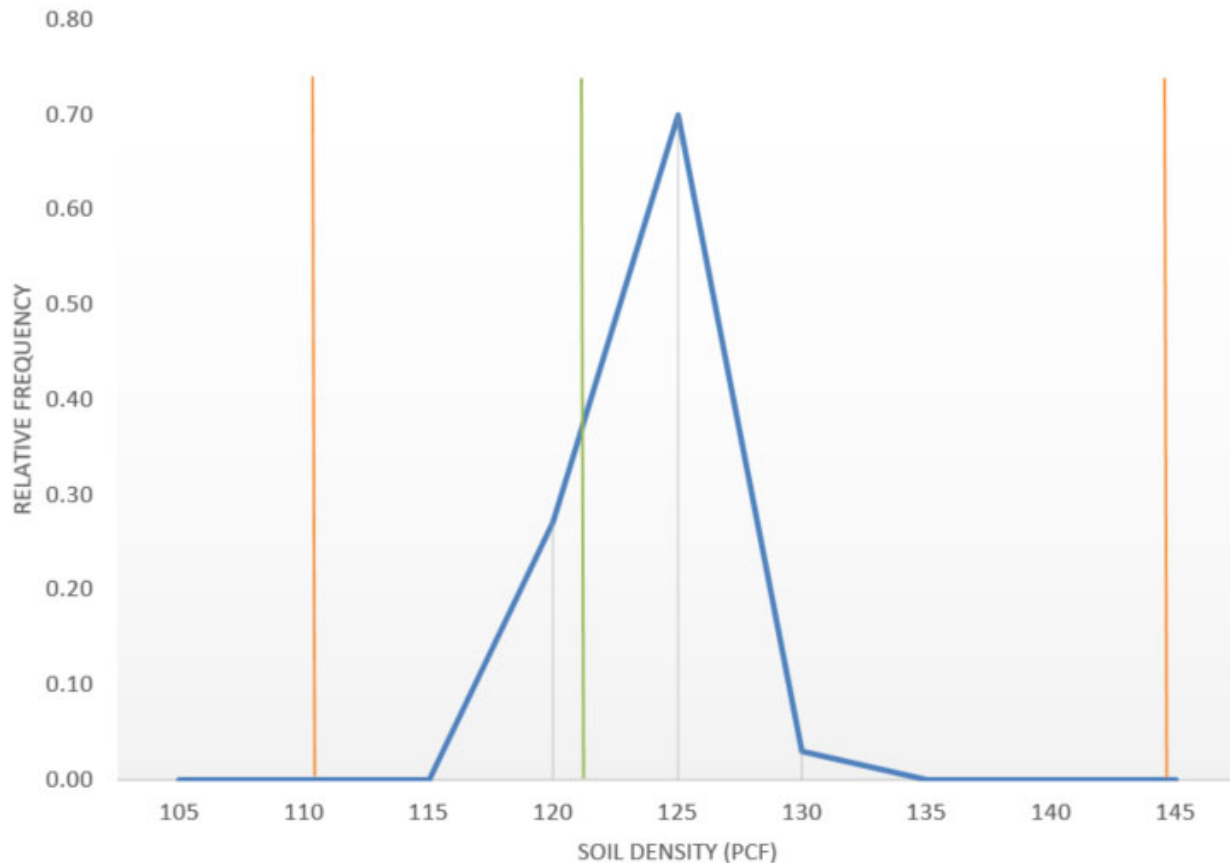
A groundwater table within the foundation depth is 3 feet below grade at the wind turbine locations as indicated by the design foundation drawing.

A total of 14 piezometers were installed in November and December 2020. The groundwater data collected between December 2020 and January 2021 appears to represent deeper water levels relative to the original geotechnical field investigation. The *Piezometer Results Summary Letter* is included as **Appendix E**.

## 4.4 Soil Backfill Loads

Construction test records were reviewed and the histogram on **Figure 4-1** below was developed that shows the average soil density at each turbine location with the average moisture content of all locations.

An average backfill density of 121 pounds per cubic foot (pcf) and densities between 110 and 145 pcf were used, including load factors of 0.9 for minimum loadings and 1.2 for maximum loadings to bracket the data from the construction test records.



**Figure 4-1 Histogram of Average Soil Density at Each Turbine with Average Site Moisture Content**

Also included in the 2021 design procedures and in accordance with ACI 336R.4 (**Technical Reference 1**), an additional effect due to soil edge friction was idealized as a “soil wedge” when performing stability and ultimate strength checks. The soil wedge is not considered for fatigue strength checks.

## 5.0 Structural Assessment

Barr conducted a structural assessment of the existing foundations in accordance with 2021 standards. The assessment considered loads for the GE 1.5SLE and GE 1.6-91/GE 1.6-97 as outlined in **Project Documents 1** and **2**, respectively.

### 5.1 Global Stability, Bearing Capacity and Stiffness

#### 5.1.1 Foundation Summary

The foundations passed most of the global checks as summarized in the **Table 5-1** below.

**Table 5-1 Summary of Global Stability, Bearing Capacity and Stiffness Assessment**

Paragraph Number	Design Item	Nobles (GE 1.6-97)	Nobles (GE 1.6-91)	Acceptance Criteria
5.1.2	Factor of Safety Against Sliding	7.6	7.4	>1.5
5.1.2	Factor of Safety Against Overturning – Check #1	1.9	1.8	>1.5
5.1.2	Factor of Safety Against Overturning – Check #2	1.1	1.1	>1.0
5.1.3	Normal Load Bearing Contact Length	97% <sup>A</sup>	99% <sup>A</sup>	>100%
5.1.3	Extreme Load Bearing Contact Length	60%	56%	>50%
5.1.4	Normal Load Bearing Capacity Utilization Ratio	0.50	0.48	<1.00
5.1.4	Extreme Load Bearing Capacity Utilization Ratio	0.55	0.52	<1.00
5.1.4	Abnormal Load Bearing Capacity Utilization Ratio	0.53	0.59	<1.00
5.1.4	Seismic Load Bearing Capacity Utilization Ratio	0.31	0.30	<1.00
5.1.5	Rotational Stiffness	85 GN-m/rad	90 GN-m/rad	>50 GN-m/rad (GE 1.6-97m) >30 GN-m/rad (GE 1.6-91m)
5.1.6	Translational Stiffness	1.5 GN/m	1.6 GN/m	>1.0 GN/m

<sup>A</sup> Value less than the 100% is acceptable based on cyclic degradation testing performed (see Section 5.1.3)

#### 5.1.2 Overturning and Sliding Stability

Barr computed a factor of safety against overturning on service level extreme loads in accordance with **Standards 1** and **4**.

#### 5.1.3 Foundation Base Soil Bearing Contact Length

Barr computed the bearing contact length with the soils beneath the foundation under normal and extreme loads in accordance with **Technical Reference 4**. The bearing contact length under normal loadings was found to be insufficient. Barr calculated a contact area of 97% and 99%, which are less than the requirement of 100%.

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In response to this, a geotechnical investigation was completed to evaluate the potential for foundation “gapping” to have detrimental cyclic degradation impacts on the supporting soils. For a number of loading cycles calculated to be equivalent to 31 years of turbine operation (11 years baseline turbine plus 20 years of repower turbine), the soil samples were found to not be susceptible to degradation. The *Cyclic Degradation Geotechnical Evaluation Letter* is included as **Appendix E**.

#### **5.1.4 Foundation Bearing Capacity**

Based on the original geotechnical investigation, an allowable bearing capacity of 3,300 psf was used in the evaluation for normal loads on the foundation. Bearing capacity utilization factors for normal operating loads, normal extreme loads, abnormal extreme loads, and seismic plus operating loads were calculated in accordance with **Standard 1**. The bearing capacity utilization factor is defined as the applied bearing pressure divided by the allowable bearing pressure.

#### **5.1.5 Foundation Stiffness**

Foundation stiffness requirements are included in the repower turbine load documents, **Project Documents 1** and **2**. It lists the minimum required foundation rotational stiffness as either 30 or 50 GigaNewton meters per radian (GN-m/rad) and a minimum foundation lateral stiffness of 1.0 GigaNewtons per meter (GN/m).

The stiffness properties of the soil were the same as assumed in the original foundation design. A strain reduction factor in accordance with Fahey (**Technical Reference 5**) was applied to be consistent with 2021 design procedures. Additionally, the area of the foundation used within the stiffness calculation was reduced based on the contact area discussed in Section 5.1.3.

## 5.2 Tower/Foundation Connection Ultimate Strength

In the following sections, the “strength utilization ratio” is defined as the effect of the design load divided by the design strength. The strength utilization ratio must be less than 1.0 for the design element to pass.

### 5.2.1 Foundation Summary

The tower/foundation connection passed all strength design checks as summarized in **Table 5-2** below.

**Table 5-2 Summary of Tower/Foundation Connection Ultimate Strength Assessment**

Paragraph Number	Design Item	Nobles (GE 1.6-97)	Nobles (GE 1.6-91)	Acceptance Criteria
5.2.2	Anchor Bolt Strength Utilization Ratio	0.70	0.69	<1.00
5.2.3	Bottom Flange Bearing on Pedestal Concrete Strength Utilization Ratio	0.81	0.80	<1.00
5.2.4	Grout Strength Utilization Ratio	0.91	0.90	<1.00
5.2.5	Embedment Plate/Anchor Bolt Pull-out Strength Utilization Ratio (concrete only)	0.73	0.71	<1.00
5.2.6	Embedment Plate Strength Utilization Ratio	0.79	0.79	<1.00

### 5.2.2 Anchor Bolt Strength

The anchor bolt strength was checked in accordance with AISC (**Code 1b**).

The tension in the anchor bolts is also critical to ensuring that the anchor bolts have the capacity to resist fatigue due to the oscillating nature of the tension applied by the wind turbine tower to the bolts. A common maintenance practice is for 10% or so of the anchor bolts on each foundation to be checked for adequate tension at an annual or some other interval and for all bolts to be re-tightened if any bolts fail the tension check.

### 5.2.3 Bottom Flange Bearing on Concrete Pedestal Strength

Bottom flange bearing on the concrete pedestal was checked in accordance with ACI 318 (**Code 1a**).

### 5.2.4 Grout Strength

Bottom flange bearing on the grout was checked in accordance with ACI 318 (**Code 1a**).

### 5.2.5 Embedment Plate/Anchor Bolt Pull-out Strength

The original design considered the pull-out strength of the embedment plate/anchor bolt connection to be provided by the shear strength of a presumed concrete failure cone and the exterior vertical bars. While current design procedures rely only on vertical reinforcing steel surrounding the embedment plate to provide strength, ignoring the strength of concrete, AWEA/ASCE (**Standard 1**) also states that procedures which use only concrete resistance to design the connection can also be used.

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For this 2021 assessment, the 2009 procedure was modified, considering two known connection failures that have occurred in foundations designed according to these older procedures. In these failures, only an outside failure cone around the outside circumference of the embedment plate formed, rather than the originally assumed outside and inside failure cone about the inner and outer width of the embedment plate. Since the extreme design load was not known to occur in these failure instances and the failures were thought to have occurred due to fatigue, an ultimate design strength in accordance with PCI (**Technical Reference 7**), was used for this assessment, which was consistent with that used in some mid-2000s design practices. The strength provided by the vertical bars to the connection was conservatively ignored.

### **5.2.6 Embedment Plate Strength**

The embedment plate strength was checked in accordance with AISC (**Code 1b**).



## 5.3 Reinforced Concrete Ultimate Strength

Reinforced concrete ultimate strength design checks were performed in accordance with the requirements of ACI 318 (**Code 1a**). Size effects in concrete shear strength were also considered (**Technical Reference 3**). The design check procedures also used **Technical Reference 1** and **4** for calculation of soil bearing and friction resistances. Characteristic design loads were factored in accordance with ACI 318 (**Code 1a**), AWEA/ASCE (**Standard 1**), and IEC (**Standard 4**).

### 5.3.1 Foundation Summary

The foundation passed the ultimate strength design checks as summarized in **Table 5-3**.

**Table 5-3 Summary of Reinforced Concrete Ultimate Strength Assessment**

Paragraph Number	Design Item	Nobles (GE 1.6-97)	Nobles (GE 1.6-91)	Acceptance Criteria
5.3.2	Footing Concrete One-Way Shear ACI Strength Utilization Ratio	0.54	0.51	<1.00
5.3.2	Footing Concrete One-Way Shear ACI 318-19 Strength Utilization Ratio	0.94	0.88	<1.00
5.3.3	Footing Bottom Reinforcing Moment Strength Utilization Ratio	0.66	0.63	<1.00
5.3.3	Footing Top Reinforcing Moment Strength Utilization Ratio	1.04 <sup>A</sup>	1.04 <sup>A</sup>	<1.00
5.3.4	Footing Two-Way Shear Strength Utilization Ratio	0.34	0.34	<1.00

<sup>A</sup> Small exceedance of criteria is considered acceptable for this parameter of top reinforcing moment

### 5.3.2 Footing Concrete One-Way Shear Strength

Barr calculated the footing concrete one-way shear strength in accordance with the requirements of ACI 318 (**Code 1a**). Barr also checked the one-way shear strength in accordance with **Technical Reference 3** to further account for size effects associated with the thickness of the footing.

### 5.3.3 Footing Moment Strength

The top and bottom reinforcing bars were checked in accordance with ACI 318 (**Code 1a**) in orthogonal directions (east-west, north-south), at 45 degrees to the orthogonal, and at the cut-off locations. Moment strength was checked across the entire foundation width at the critical section as specified in section 15.4 of ACI 318 (**Code 1a**).

### 5.3.4 Footing Two-Way Shear Strength

The two-way shear strength of the footing was checked in accordance with ACI 318 (**Code 1a**). The fraction of the unbalanced moment that could be carried by the flexural reinforcing steel was calculated, with the remainder carried by concrete shear.

## 5.4 Reinforced Concrete Fatigue Strength

Reinforced concrete fatigue strength was checked in accordance with provisions contained in the DNV Offshore Standard (**Standard 2**), thus meeting the requirements of AWEA/ASCE (**Standard 1**). A duration of 20 years is the standard design life prescribed by IEC (**Standard 4**), and that which GE assumed when forming the fatigue loading. The fatigue strength assessment described herein added additional cycles/time equal to 11 years for the GE 1.5SLE to the 20 years for GE 1.6-91/GE 1.6-97 fatigue cycles/time.

### 5.4.1 Foundation Summary

The foundation passed the fatigue strength design checks as summarized in **Table 5-4**.

**Table 5-4 Summary of Reinforced Concrete Fatigue Strength Assessment**

Paragraph Number	Design Item	Nobles (GE 1.6-97)	Nobles (GE 1.6-91)	Acceptance Criteria <sup>A</sup>
N/A	Evaluated Duration	31 Years		N/A
5.4.2	Footing Concrete Compression Cumulative Damage	0.00	0.00	<1.00
5.4.3	Footing Concrete One-Way Shear Cumulative Damage	0.00	0.00	<1.00
5.4.4	Grout Bearing Cumulative Damage	0.00	0.00	<1.00
5.4.5	Pedestal Concrete Bearing Cumulative Damage	0.41	0.07	<1.00
5.4.5	Pedestal Bursting Steel Cumulative Damage	NA	NA	<0.50
5.4.6	Embedment Plate/Anchor Bolt Pull-out Cumulative Damage (concrete only)	0.00	0.00	<1.00
5.4.7	Bottom Reinforcing Cumulative Damage	0.00	0.00	<0.50
5.4.8	Top Reinforcing Cumulative Damage	0.48	0.48	<1.00

<sup>A</sup> Values based on categorization of "Above splash zone" region within DNV-OS-C502

### 5.4.2 Footing Concrete Fatigue Strength in Beam Compression

Footing concrete in beam compression was checked in accordance with DNV (**Standard 2**).

### 5.4.3 Footing Concrete Fatigue Strength in One-Way Shear

Footing concrete fatigue strength in one-way shear was checked using provisions contained in DNV (**Standard 2**). One-way shear across the entire foundation width was checked at the critical section for shear as specified in section 11.1.3.1 of ACI 318 (**Code 1a**). It is industry standard to assume that a portion of the cross section carries the load from the entire cross section because the outer portions of the foundation have reduced stiffness and redistribution of load is generally not assumed for fatigue loading. Barr assumed an effective foundation width of the embedment ring diameter plus three times the footing thickness for this check to resist all one-way shear required for static equilibrium. Barr is not aware of any standard that specifically prescribes an effective foundation width to be used in this calculation. The width selected by Barr is specified in ACI 318 (**Code 1a**) for an analogous application related to reinforced concrete ultimate strength design.

#### 5.4.4 Grout Fatigue Strength in Bearing

Grout fatigue strength in bearing was checked using the provisions contained in DNV (**Standard 2**).

#### 5.4.5 Pedestal Concrete and Bursting Reinforcement Fatigue Strength in Bearing

Pedestal concrete fatigue strength in bearing was checked using the provisions contained in DNV (**Standard 2**).

#### 5.4.6 Fatigue Strength in Embedment Plate/Anchor Bolt Pull-Out

Pull-out fatigue strength was calculated using a fatigue design strength of 130 psi calculated by Barr and applying the provisions contained in DNV, **Standard 2**, and found to be acceptable. The source for the fatigue design strength value is described below. The vertical bars connecting the footing to the pedestal were conservatively ignored and were assumed to not contribute to the fatigue strength of the connection, as stipulated by **Standard 1**.

Barr derived the design fatigue pull-out strength for the pullout cone using data related to two known failure cases. In both these cases the embedment plate anchor bolt pulled out a cone of failure from the middle of the footing and the tower collapsed. One case happened after 8 years of operation and the other after 12 years of operation.

Using the fatigue rain flow spectrum for the wind turbine and 8 years of load cycles, a fatigue pull-out strength was derived which produced a cumulative damage of 1 and the presumed failure. The value calculated was 175 psi. For the fatigue check, this fatigue pull-out strength was further divided by a material coefficient of 1.35 to produce a design fatigue pull-out strength of 130 psi. The value of 130 psi correlates well with the 133 psi design shear strength of concrete as prescribed by DNV (**Standard 2**), and 142 psi design ultimate shear strength of concrete as prescribed by ACI 318, (**Code 1a**).

#### 5.4.7 Bottom Reinforcing Fatigue Strength in Tension

Bottom reinforcing fatigue strength was checked using provisions contained in DNV (**Standard 2**). Moment strength was checked across the entire foundation width at the critical section as specified in section 15.4 of ACI 318 (**Code 1a**). The effective foundation width resisting all bending moment required for static equilibrium in this check was the same width that was used for shear (embedment ring diameter plus three times the footing thickness).

#### 5.4.8 Top Reinforcing Fatigue Strength in Tension

Top reinforcing fatigue strength was checked using provisions contained in DNV (**Standard 2**). Moment strength was checked across the entire foundation width at the critical section as specified in section 15.4 of ACI 318 (**Code 1a**) as well as the location of bending in the reinforcement. The effective foundation width resisting all bending moment required for static equilibrium in this check was the same width that was used for shear (embedment ring diameter plus three times the footing thickness).

## 6.0 Recommendations

The foundations passed all design checks. One item of note is that industry design practices for the anchorage of the tower to the foundation are different today than they were when the original designs were completed. Current design practice is to include vertical reinforcing steel bars surrounding the embedment ring and anchor bolt assembly to resist the entire pullout force exerted on the foundation. In the early and mid-2000's the anchorage was often designed to rely on the strength of a presumed shear failure surface within the concrete. This type of anchorage is generally allowed by **Standard 1**, although specific design measures are not provided and an explicit method to evaluate the fatigue resistance of this connection is not prescribed by any design code. For this evaluation, Barr developed a calculation procedure based on a design fatigue strength for this type of anchorage based on an analogous design formula and Barr's knowledge of historical connection performance. This alternative assessment procedure determined that the ultimate and fatigue pull-out strength met design requirements.

To validate the structural assessment, Barr performed instrumentation, monitoring, and visual condition inspection within a sample size of the existing foundations to verify that the pedestal, foundation, and connection are in good condition.

The *Wind Turbine Foundation Visual and Coring Inspection Summary Report* is included in **Appendix B**. The report summarizes the evaluation of 28 wind turbine foundations via the retrieval of cores extending from the surface of the footing towards the embedment plate under the pedestal. Some level of potential shear cracking or regions of poor consolidation were observed in cores within the following foundations:

- 16 (both cores),
- 39 (one core), and
- 47 (one core).

This type of cracking could be expected from the development of a pullout cone failure mechanism if the cracking plane crosses the axis of the core. No cracks were observed in the cores obtained from within the other 25 foundations.

A *Foundation Structural Health Monitoring Summary Report* is also included in **Appendix C**. The monitoring efforts identified the following:

- 13 of the 14 foundations monitored (5, 6, 13, 25, 35, 40, 47, 58, 75, 77, 84, 99, and 104) appear to be functioning above the GE 1.6-91 repower turbine minimum rotational stiffness value of 30 GN-m/rad.
- All but two foundations monitored (6 and 73) appear to be functioning above the GE 1.6-97 repower turbine minimum rotational stiffness value of 50 GN-m/rad.

Barr's opinion is that the foundations in their current configuration are adequate to support the new GE turbines with 20 years of fatigue cycles/time given the following recommendations:

- 
- (1) Perform structural health monitoring within 1 year on the three foundations (16, 39 and 47) with some level of cracking or poor consolidation identified within the cores.
  - (2) Perform structural health monitoring again in 1 year on the two foundations (6 and 73) with measured average rotational stiffness values below 50 GN-m/rad.

The assessment described herein is based on the complete load documents provided by GE. Barr's analysis and calculations rely on the foundation loading provided by GE and are dependent on the load data being correct. The load data cannot be independently confirmed. The actual loads experienced by the foundation will likely vary depending on weather conditions and equipment performance.

The tension in the anchor bolts is also critical to ensuring that the anchor bolts have the capacity to resist fatigue due to the oscillating nature of the tension applied by the wind turbine tower to the bolts. A common maintenance practice is for 10% or so of the anchor bolts on each foundation to be checked for adequate tension at an annual or some other interval and for all bolts to be re-tightened if any bolts fail the tension check.

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## 7.0 Limitations of Analysis

This report contains opinions made by Barr, using its professional judgement and reasonable care. Use or reliance upon this report is subject to the following conditions:

1. The report being read in the context of and subject to the terms of the agreement between Barr and Xcel, including the methodologies, procedures, techniques, assumptions, and other relevant terms or conditions that were specified or agreed therein.
2. The report being read as a whole, with sections or parts hereof read or relied upon in context.
3. The report is based on information made available to Barr by Xcel or by third parties and, unless stated otherwise in this report, Barr has not verified the accuracy, completeness, or validity of such information, makes no representations regarding its accuracy and assumes no liability in connection therewith.
4. GE has provided a complete load document that was used for the evaluation. Barr's analysis and calculations rely on the foundation loading provided by GE and are dependent on the GE load data being correct. The GE load data cannot be independently confirmed. The actual loads experienced by the foundation most likely were and will be different from those contained in the loading document, depending on weather conditions, equipment performance, and the occurrence of events not considered in the loads provided by GE.
5. The conclusions and opinions expressed in this report are based on a review of the documents listed in **Section 8.2**, as well as our education, training, and experience as licensed professional engineers. These conclusions and opinions are based on the information that has been provided to us at this time and may be modified or supplemented if new information becomes available.

## 8.0 References

### 8.1 Project Documents

1. GE Renewable Energy, "Foundation Load Specification for Wind Turbine Generator Systems", Nobles RePower, Minnesota, GE Project 1228754, 1.6-97, 60 Hz, 80m Hub Height, Revision 02, February 26, 2021.
2. GE Renewable Energy, "Foundation Load Specification for Wind Turbine Generator Systems", Nobles RePower, Minnesota, GE Project 1228754, 1.6-91, 60 Hz, 80m Hub Height, Revision 02, March 9, 2021.
3. Barr Engineering Co., Nobles Wind Project, GE Spread Footing Foundation, Drawing No. S-01, Revision 1, June 10, 2010.

### 8.2 Codes, Standards and Technical References

#### 8.2.1 Codes

1. International Code Council Inc., International Building Code, 2018.
  - a. American Concrete Institute, ACI 318-14, Building Code Requirements for Structural Concrete, 2014.
  - b. American Institute of Steel Construction Inc., Steel Construction Manual, 2016.
  - c. American Society of Civil Engineers, ASCE 7-16, Minimum Design Loads for Buildings and Other Structures, 2016.

#### 8.2.2 Standards

1. American Wind Energy Association, American Society of Civil Engineers, ASCE/AWEA RP2011, Recommended Practice for Compliance of Large Land-Based Wind Turbine Support Structures, December 2011.
2. Det Norske Veritas, Offshore Standard DNV-OS-C502, Offshore Concrete Structures, February 2018.
3. Germanischer Lloyd, GL Wind Guidelines, Edition 2010.
4. International Electrotechnical Commission, Wind Turbine Generator Systems – Part 1: Safety Requirements, 3rd edition, 2005.

#### 8.2.3 Technical References

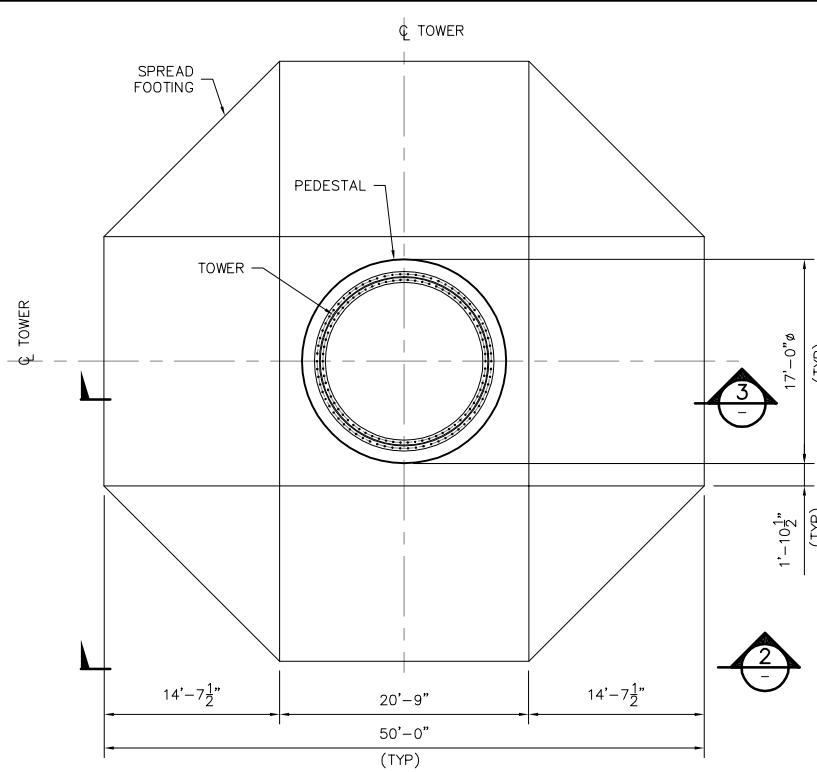
1. American Concrete Institute, ACI 336R.4-XX, "Guide for Analysis of Spread Footings by the Strength Design Method", Draft 2008.

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2. Arya, S. O'Neill, M., Pincus, G., Design of Structures and Foundations for Vibrating Machines, Gulf Publishing Company, 1984.
  3. American Concrete Institute, ACI 318-19, Footing Shear Provisions, Building Code Requirements for Structural Concrete, 2019.
  4. Det Norske Veritas Copenhagen and Wind Energy Department, Riso National Laboratory, Guidelines for Design of Wind Turbines, 2nd Edition, 2002.
  5. Fahey M., "Soil stiffness values for foundation settlement analysis", Proc. 2nd Int. Conf. on Pre-Failure Deformation Characteristics of Geomaterials, Torino, Vol. 2, 1325-1332, 1999.
  6. Potyondy, J.G., "Skin Friction between Various Soils and Construction Materials", Geotechnique, December 1961.
  7. Precast/Prestressed Concrete Institute, PCI Design Handbook, 5th Edition, 1999.
  8. Nowak, Andrzej S., and Collins, Kevin R., Reliability of Structures, 2000.
  9. Squeglia, Nicholas L., "Zero Acceptance Number Sampling Plans, 5<sup>th</sup> Edition, 2008.

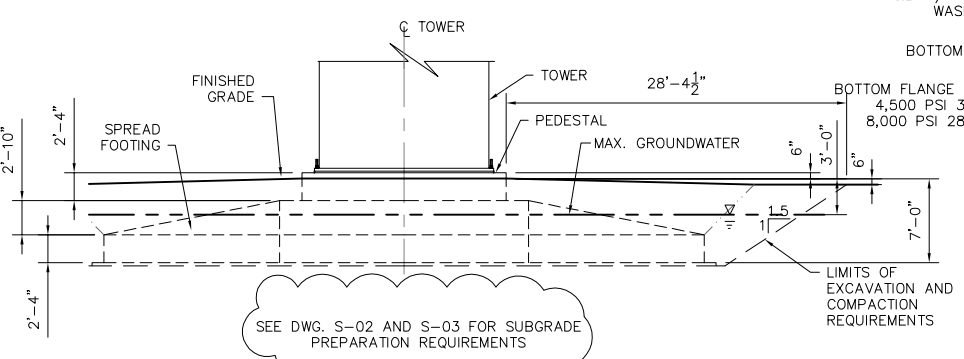
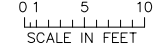


## **Appendix A**

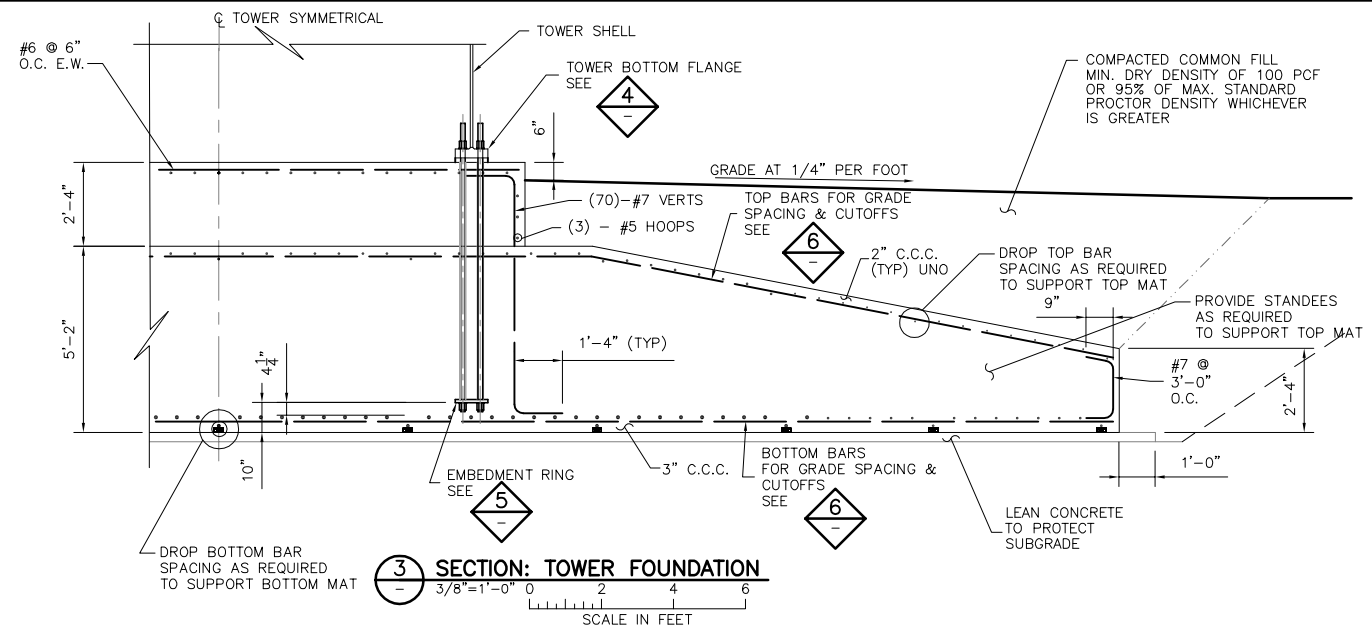
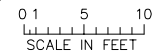
### **Foundation Design Drawings and Assessment Calculations**



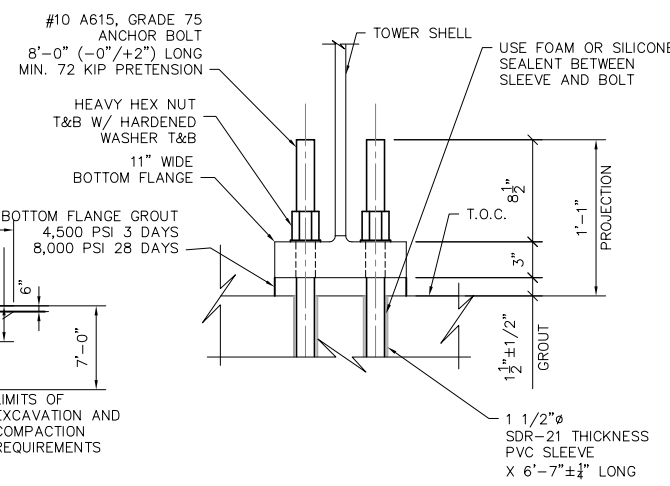
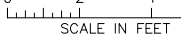
**1 PLAN: TOWER FOUNDATION**  
1/8"=1'-0"



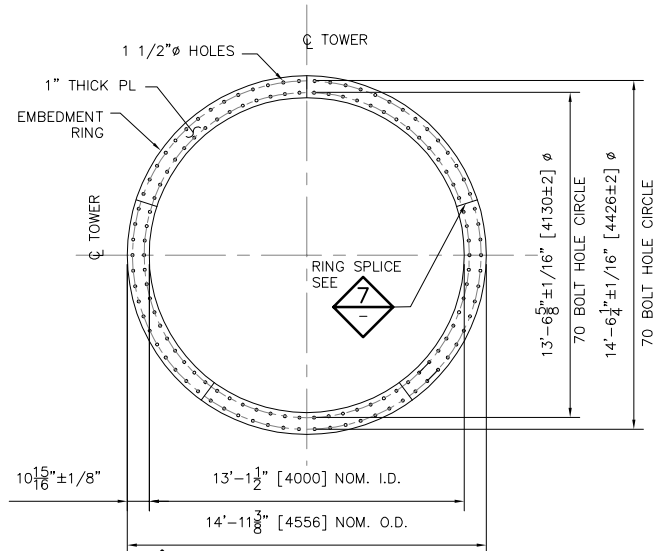
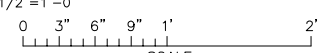
**2 ELEVATION: TOWER FOUNDATION**  
1/8"=1'-0"



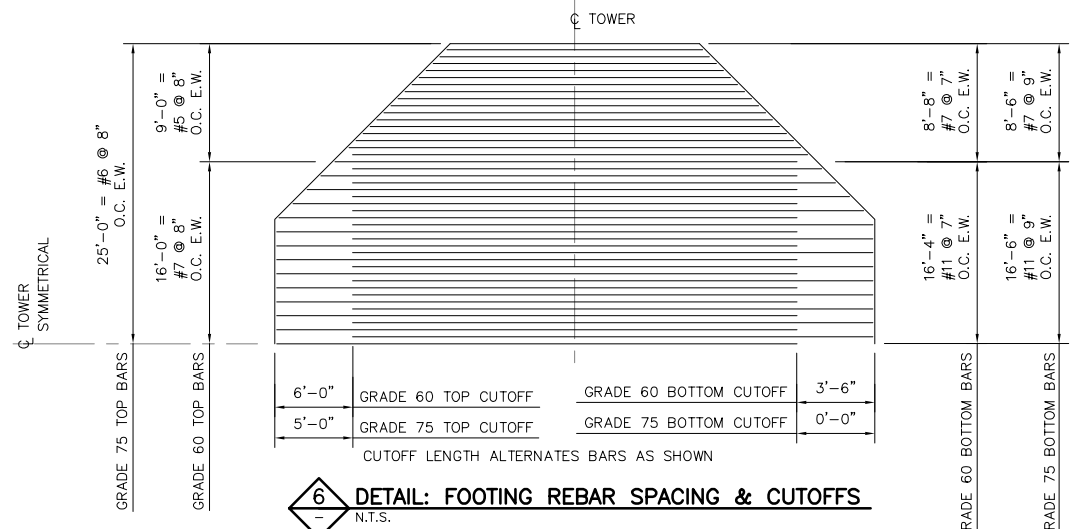
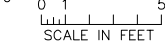
**3 SECTION: TOWER FOUNDATION**  
3/8"=1'-0"



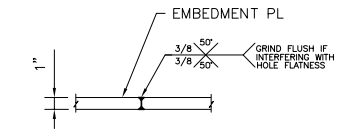
**4 DETAIL: TOWER BOTTOM FLANGE**  
1 1/2"=1'-0"



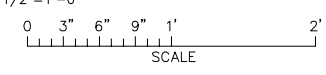
**5 DETAIL: EMBEDMENT RING**  
1/4"=1'-0"



**6 DETAIL: FOOTING REBAR SPACING & CUTOFFS**  
N.T.S.



**7 DETAIL: EMBEDMENT PL WELD**  
1 1/2"=1'-0"



**BUILDING AND DESIGN CODES:**  
INTERNATIONAL BUILDING CODE 2006, INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS.  
BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE, ACI 318, 2005, AMERICAN CONCRETE INSTITUTE.

**WIND TURBINE AND TOWER:**  
MANUFACTURER: GE ENERGY  
MODEL: GE1.5 SLE  
POWER OUTPUT: 1.5 MW  
TURBINE HUB HEIGHT: 80m  
ROTOR DIAMETER: 77M

**DESIGN SERVICE LOADS:**  
UNFACTORED SERVICE LOADS DUE TO EXTREME WIND CONDITION CLASS IEC-IIA (CRITICAL)

OVERTURNING MOMENT,  $M_{xy} = 34,908 \text{ kN-m} = 25,747 \text{ ft-kips}$   
HORIZONTAL BASE SHEAR,  $H_{xy} = 523 \text{ kN} = 118 \text{ kips}$   
VERTICAL TOWER LOAD,  $W_z = 1,845 \text{ kN} = 415 \text{ kips}$

**FOUNDATION DESIGN DATA:**  
MIN. FACTOR OF SAFETY AGAINST OVERTURNING: >1.5  
MIN. FACTOR OF SAFETY AGAINST SLIDING: >1.5  
MIN. FACTOR OF SAFETY AGAINST BEARING CAPACITY FAILURE: >2.26

**REFERENCE DOCUMENTS:**

- GE ENERGY, "FOUNDATION DATA FOR WIND TURBINE GENERATOR SYSTEMS, GE 1.5SLE MTS, 61.4-85M HH, IEC TCIA WITH REDUCED GUST (V<sub>50</sub>=55 M/S), LM 37.3P2 50& 60 Hz, GE37C 60 Hz", REVISION, DATED JULY 1, 2009.
- BARR ENGINEERING COMPANY, "GEOTECHNICAL ENGINEERING REPORT, NOBLES WIND PROJECT, NOBLES COUNTY, MINNESOTA", SEPTEMBER 2009.

**MIN. 28-DAY COMPRESSIVE STRENGTH OF CONCRETE:**  
5000 PSI

**MIN. YIELD POINT STRENGTH OF REINFORCING BAR:**  
60 KSI UNLESS NOTED 60 KSI OR 75 KSI FOR FOOTING

**MIN. STRENGTH OF ANCHOR BOLTS:**  
TENSILE STRENGTH 100 KSI, YIELD STRENGTH 75 KSI

**MIN. COMPRESSIVE STRENGTH OF NON-SHRINK GROUT:**  
8,000 PSI @ 28 DAYS, 4,500 PSI @ 3 DAYS

**MIN. YIELD POINT STRENGTH OF EMBEDMENT PLATE:**  
36 KSI

**TOTAL NET VOLUME OF CONCRETE:**  
322 CUBIC YARDS

**ESTIMATED WEIGHT OF REINFORCING STEEL:**  
21.7 TONS ALL GRADE 60  
18.3 TONS WITH GRADE 75 FOOTING BARS

**FOR CONSTRUCTION**

**READ DRAWING IN CONJUNCTION WITH DRAWING S-02 TECHNICAL SPECIFICATIONS AND SUBMITTALS**

**ABBREVIATIONS:**

B.O.	BOTTOM OF	O.C.	ON CENTER
C.C.C.	CLEAR CONCRETE COVER	O.D.	OUTSIDE DIAMETER
CL	CENTER LINE	R	RADIUS
EL.	ELEVATION	T&B	TOP AND BOTTOM
E.W.	EACH WAY	T.O.C.	TOP OF CONCRETE
EX.	EXISTING	TYP	TYPICAL
I.D.	INSIDE DIAMETER	UNO	UNLESS NOTED OTHERWISE
MIN.	MINIMUM	W/	WITH
NOM.	NOMINAL	Ø	DIAMETER

**CONFIDENTIAL**

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NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
A	CAK	-	CAK	11-NOV-09	INITIAL RELEASE
B	CAK	-	CAK	11-NOV-09	OPTION FOR GRADE 75 FOOTING BARS & ASSUMED GROUNDWATER LEVEL
C	CAK	MBJ	CAK	16-DEC-09	FOR CONSTRUCTION
D	CPB	MBJ	CAK	01-JUN-10	REVISED GRADE SLOPE AND DISTANCE ON SECTIONS 2 AND 3

I HEREBY CERTIFY THAT THIS ENGINEERING DOCUMENT WAS PREPARED BY ME OR UNDER MY DIRECT PERSONAL SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNATURE: *Chris Kopchynski*  
PRINTED NAME: CHRISTOPHER A. KOPCHYNSKI  
DATE: 16-DEC-09 REG. NO.: 21869

CLIENT	11/11/09	11/11/09	12/16/09	06/01/10				
BID								
CONSTRUCTION								

Project Office:  
**BARR ENGINEERING CO.**  
4700 WEST 77TH STREET  
MINNEAPOLIS, MN.  
55435-4803

Corporate Headquarters:  
Minneapolis, Minnesota  
Ph: 1-800-632-2277  
Fax: (952) 832-2601  
www.barr.com

Scale	AS SHOWN
Date	11-NOV-09
Drawn	CAK
Checked	MBJ
Designed	CAK
Approved	CAK

**M.A. MORTENSON COMPANY**  
MINNEAPOLIS, MINNESOTA

<b>NOBLES WIND PROJECT</b> NOBLES COUNTY, MINNESOTA		BARR PROJECT No. <b>23/53-1002</b>
<b>GE SPREAD FOOTING FOUNDATION</b> PLAN, ELEVATION, SECTION & DETAILS		CLIENT PROJECT No. -
DWG. No. <b>S-01</b>	REV. No. <b>1</b>	

**1.0 EXCAVATION, SUBGRADE PREPARATION, BACKFILL, & COMPACTION**

- A. GENERAL**
- ENSURE FOUNDATION SITE IS EXCAVATED AND GRADED IN ACCORDANCE WITH THE DRAWINGS.
  - ENSURE UNIFORM BEARING CONDITIONS ARE PROVIDED FOR THE FOUNDATION. DO NOT PLACE THE FOUNDATION ON VARIABLE OR DISTURBED SOIL CONDITIONS. UNIFORMITY SHALL BE VERIFIED BY USE OF HAND PENETROMETER TESTS.
- B. SUBMITTALS**
- SUBMIT GRAIN SIZE ANALYSIS, NATURAL MOISTURE CONTENT, AND STANDARD PROCTOR MAXIMUM DRY DENSITY TEST DATA FOR COMMON FILL SOIL MATERIALS.
  - SUBMIT COMPACTION TEST RESULTS FOR FILL PLACED OVER THE FOUNDATION INDICATING LOCATION OF TEST, DRY DENSITY, AND MOISTURE CONTENT OF PLACED FILL.
  - SUBMIT SUBGRADE INSPECTION REPORT FOR EACH FOUNDATION COMPLETED BY A GEOTECHNICAL ENGINEER.
  - SUBMIT GROUNDWATER CONTROL PLAN.
- C. PRODUCTS**
- LEAN CONCRETE:** CONTAINING ASTM C150, TYPE I CEMENT.
  - COMMON FILL:** SHALL CONSIST OF SUITABLE MATERIALS EXCAVATED FROM THE FOUNDATION SITE OR IMPORTED AS NECESSARY. ADDITIONAL CRUSHING AND SCREENING MAY BE REQUIRED TO PROCESS THE MATERIAL TO THE SPECIFICATION REQUIREMENTS.
    - MATERIALS BACKFILLED WITHIN 1 FOOT OF THE PEDESTAL SHALL BE FINE MATERIAL WITH A PARTICLE SIZE NO GREATER THAN 3".
    - MATERIAL BACKFILLED OUTSIDE OF THIS AREA BUT OVER THE FOUNDATION SHALL HAVE A PARTICLE SIZE NO GREATER THAN 6".
    - MATERIALS BACKFILLED OUTSIDE THIS AREA MAY CONSIST OF ALL OTHER EXCAVATED MATERIALS PROVIDED THEY CAN MEET THE DENSITY REQUIREMENTS.
    - ALL BACKFILL MUST BE PLACED USING METHODS THAT WILL PREVENT VOIDS FROM OCCURRING.
- D. EXECUTION**
- SCRAPE TOPSOIL FROM THE PLAN AREA AND STORE IN AN OWNER DESIGNATED AREA. THE TOPSOIL SHALL BE USED FOR SITE RESTORATION.
  - EXCAVATE SOILS TO THE LIMITS INDICATED ON DRAWING S-01 AND S-03 USING TECHNIQUES (E.G. A SMOOTH-EDGED EXCAVATOR BUCKET) THAT WILL MINIMIZE DISTURBANCE TO THE SUBGRADE. CONTRACTOR IS RESPONSIBLE FOR CONTROL OF SURFACE WATER AND/OR GROUNDWATER FLOWS INTO THE EXCAVATION.
  - NOT USED.
  - PRIOR TO PLACING PROTECTIVE LEAN CONCRETE SURFACE, HAVE A PROFESSIONAL GEOTECHNICAL ENGINEER OR A PERSON DESIGNATED AND UNDER THE DIRECT SUPERVISION OF THIS ENGINEER TO INSPECT THE SUBGRADE CONDITIONS AND RECORD THE SOIL TYPE ENCOUNTERED, GROUNDWATER CONDITIONS OR OTHER SUBSURFACE CONDITIONS. CONFIRM UNIFORM BEARING CONDITIONS ARE PROVIDED BY USE OF HAND PENETROMETER TESTS. PERFORM A MINIMUM OF 33 TESTS. SHOULD TESTS VARY BY MORE THAN PLUS OR MINUS 0.5 TSF IMMEDIATELY REPORT THE RESULTS TO THE FOUNDATION ENGINEER. CHECK THAT OBSERVATIONS TAKEN ARE CONSISTENT WITH THE OBSERVATIONS CONTAINED IN THE REFERENCE GEOTECHNICAL DOCUMENTS. SUBMIT A SUBGRADE INSPECTION REPORT COMPLETED BY THE GEOTECHNICAL ENGINEER.
  - IF SOIL CONDITIONS ARE ENCOUNTERED THAT ARE NOT CONSISTENT WITH THE REFERENCED GEOTECHNICAL DOCUMENTS. OBTAIN WRITTEN INSTRUCTIONS FROM THE FOUNDATION ENGINEER AS TO THE MEANS OF CORRECTION TO BE UNDERTAKEN. OBTAIN WRITTEN CONFIRMATION FROM THE GEOTECHNICAL ENGINEER THAT THE SPECIFIED ACTIONS WERE COMPLETED.
  - FOR PROTECTION OF THE SUBGRADE AND ESTABLISHMENT OF A WORKING SURFACE, PLACE LEAN CONCRETE FILL AS INDICATED ON THE DRAWING. IT IS RECOMMENDED THAT THE LEAN CONCRETE FILL BE PLACED AS LEVEL AS PRACTICAL TO FACILITATE PLACEMENT OF THE REINFORCING STEEL AND EMBEDMENT RING. THE LEAN CONCRETE FILL MUST BE THICK ENOUGH TO SUPPORT THE REINFORCING STEEL AND THE ANCHOR BOLT CAGE.
  - BACKFILL AND COMPACTION: PLACE AND COMPACT COMMON FILL MATERIALS TO THE LIMITS, DEPTH AND DRY DENSITY INDICATED ON DRAWINGS S-01 AND S-03. PLACE FILL IN MAXIMUM LOOSE LIFTS OF 12 INCHES OR LESS TO ACHIEVE THE SPECIFIED DENSITY. BACKFILL MAY BE PLACED WHEN THE FOOTING AND PEDESTAL HAVE REACHED A COMPRESSIVE STRENGTH OF 2000 PSI.
  - GRADE THE SITE IN ACCORDANCE WITH DRAWINGS TO PREVENT WATER FROM PONDING OVER THE FOUNDATION WHILE MAINTAINING AT LEAST THE MINIMUM DEPTH OF FILL SPECIFIED ON THE DRAWINGS. RESTORE THE SITE IN ACCORDANCE WITH OWNER REQUIREMENTS.
- E. TESTING AND INSPECTION**
- FOR EVERY 2500 CUBIC YARDS OF PLACED COMMON FILL, OBTAIN SAMPLES OF COMMON FILL MATERIALS AND PERFORM GRAIN SIZE ANALYSIS, MOISTURE CONTENT, PROCTOR AND/OR RELATIVE DENSITY TESTS.
  - FOR PLACED AND COMPACTED COMMON FILLS, PROVIDE ONE DENSITY TEST PER LIFT INDICATING TEST LOCATION, DRY DENSITY AND MOISTURE CONTENT.
  - PROVIDE A SUBGRADE INSPECTION REPORT TO BE COMPLETED BY A GEOTECHNICAL ENGINEER.

**2.0 CAST-IN-PLACE CONCRETE AND STEEL REINFORCING**

- A. GENERAL**
- CONCRETE WORK SHALL BE IN COMPLIANCE WITH THE FOLLOWING CODES AND SPECIFICATIONS:
    - ACI 318-05, BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE.
    - ACI 301, STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE.
- B. SUBMITTALS**
- FOR EACH CONCRETE TYPE USED, SUBMIT FOR APPROVAL A MIX DESIGN CERTIFIED BY A PROFESSIONAL ENGINEER AND MEETING THE MINIMUM SPECIFIED REQUIREMENTS.
  - SUBMIT PRODUCT DATA FOR ADMIXTURES, POZZOLAN, AND CURING MEMBRANE USED ON THE PROJECT.
  - SUBMIT REINFORCING FABRICATION AND PLACEMENT SHOP DRAWINGS.
  - SUBMIT MILL REPORTS OF REINFORCING STEEL, CONFIRMING THE GRADE AND STRENGTH OF REINFORCING STEEL PROVIDED ON THE PROJECT.
  - SUBMIT QUALITY CONTROL FIELD TESTS OF AIR CONTENT, SLUMP, AIR TEMPERATURE, AND CONCRETE TEMPERATURE.
  - SUBMIT CONCRETE CYLINDER STRENGTH TEST RESULTS.
  - SUBMIT A PLAN FOR HOT AND COLD WEATHER PROTECTION OF CONCRETE IN ACCORDANCE WITH ACI 305-306 FOR APPROVAL BY THE FOUNDATION ENGINEER.
- C. PRODUCTS**
- REINFORCING BARS:** TO ASTM A615, GRADE 60 or 75, DEFORMED, UNCOATED.
  - CEMENT:** TO ASTM C150, TYPE I.
  - FLY ASH:** TO ASTM C618, CLASS F (IF SPECIFIED).
  - MINIMUM CEMENTITIOUS CONTENT:** IN ACCORDANCE WITH CONTRACT MIX DESIGN.
  - COARSE AGGREGATES:** TO ASTM C33, GRADATION NO: 57.
  - FINE AGGREGATES:** TO ASTM C33, GRADATION IN ACCORDANCE WITH CONTRACT MIX DESIGN.
  - AIR ADMIXTURE AND CONTENT:** TO ASTM C260, RANGING FROM 5.5% ± 1.5% FOR PEDESTAL AND FOOTING.
  - OTHER ADMIXTURES:** WATER REDUCING MIXTURE AND SUPERPLASTICIZER AS REQUIRED.
  - MAXIMUM WATER CEMENT RATIO:** 0.45.
  - 28 DAY COMPRESSIVE STRENGTH:** 5,000 PSI.
  - SLUMP:** IN ACCORDANCE WITH CONTRACT MIX DESIGN AT THE POINT OF DEPOSITION WITH THE ADDITION OF ADMIXTURES.
  - CURING MEMBRANE:** TO ASTM C309, TYPE 1 OR 2 (WHITE), CLASS A OR B.
- D. EXECUTION**
- PLACE CONCRETE IN ACCORDANCE WITH THE FOLLOWING DIMENSIONAL TOLERANCES:
    - REINFORCING PLAN SPACING: PLUS OR MINUS 2 INCHES.
    - REINFORCING VERTICAL SPACING: PLUS OR MINUS 1 INCH.
    - FOOTING CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 3 INCHES.
    - PEDESTAL CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 2 INCHES.
    - FOOTING PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 3 INCHES.
    - FOOTING THICKNESS: MINUS 0 INCHES, PLUS 3 INCHES.
    - PEDESTAL PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 2 INCHES.
    - PEDESTAL HEIGHT: MINUS 1 INCH, PLUS 0 INCHES.
    - PEDESTAL CENTERED TO WITHIN 2 INCHES RELATIVE TO FOOTING.
  - PLACE REINFORCING AT SPACING SPECIFIED MEETING THE REQUIRED DIMENSIONAL TOLERANCES.
  - PROVIDE NECESSARY CHAIRS AND STANDEES TO SUPPORT REBAR AND PREVENT MOVEMENT OR DISPLACEMENT OF THE MATS DURING PLACING OF CONCRETE.
  - REINFORCEMENT SHALL BE FREE OF LOOSE RUST, MILL SCALE, EARTH, ICE, CONCRETE, OR OTHER MATERIALS WHICH COULD PREVENT BONDING TO NEW CONCRETE.
  - SET FORMWORK IN ACCORDANCE WITH SPECIFIED DIMENSIONS AND TOLERANCES. PREVENT FORMWORK FROM DEFLECTING GREATER THAN 1 INCH DURING PLACEMENT OF CONCRETE. FORMWORK MUST BE REMOVED AFTER CONCRETE WORK IS COMPLETED.
  - PLACE CONCRETE IN ACCORDANCE WITH ACI 318. PLACE SUCCESSIVE LIFTS OF CONCRETE TO ENSURE PROPER AMALGAMATION OF CONCRETE BETWEEN SUCCESSIVE LIFTS.
  - CONSOLIDATE CONCRETE IN ACCORDANCE WITH ACI 318 PREVENTING THE FORMATION OF JOINTS, VOIDS, HONEYCOMBING OR SEGREGATION OF AGGREGATE.
  - ROUGH TROWEL FINISH TOP OF CONCRETE FOOTING USING A ROLLER SCREED.
  - PRIOR TO PLACING PEDESTAL CONCRETE, REMOVE CONCRETE SURFACE LAITANCE AND OTHER DEBRIS FROM TOP OF FOOTING.
  - TROWEL AND BROOM FINISH TOP OF PEDESTAL.
  - CURE CONCRETE FOOTING AND PEDESTAL IN ACCORDANCE WITH ACI 318. IF A CURING MEMBRANE IS USED, APPLY CURING MEMBRANE AS SOON AS BLEEDING HAS STOPPED AND FREE WATER HAS DISAPPEARED FROM THE SURFACE.
  - ALL METAL DEVICES USED TO SUPPORT FORMWORK OR TEMPORARY BRACING THAT ARE EMBEDDED IN THE FOOTING OR PEDESTAL SHALL BE REMOVED TO A DEPTH OF ONE INCH FROM THE SURFACE OF THE CONCRETE AND FILLED WITH GROUT.
- E. TESTING AND INSPECTION**
- FOR EACH FOOTING PLACED, CAST A MINIMUM OF 8 CONCRETE CYLINDERS FOR LABORATORY STRENGTH TESTING (2 CYLINDERS IN EACH OF 4 RANDOMLY SELECTED INTERVALS THROUGHOUT THE POUR) WITH TWO TESTS AT 28 DAYS (AVERAGE OF TWO CYLINDER BREAKS FOR EACH STRENGTH TEST) AND IF NECESSARY ONE AT 56 DAYS. IF 4 INCH BY 8 INCH CONCRETE CYLINDERS ARE USED, CAST A MINIMUM OF 12 CONCRETE CYLINDERS FOR EACH FOOTING PLACED AND USE THE AVERAGE OF THREE CYLINDER BREAKS FOR EACH STRENGTH TEST. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
  - FOR EACH PEDESTAL, CAST A MINIMUM OF 4 CONCRETE CYLINDERS FOR LABORATORY STRENGTH TESTING WITH ONE TEST AT 28 DAYS (AVERAGE OF TWO CYLINDER BREAKS FOR EACH STRENGTH TEST) AND IF NECESSARY ONE AT 56 DAYS. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
  - PERFORM A MINIMUM OF ONE AIR TEST AND A MINIMUM OF ONE SLUMP TEST PER SET OF CYLINDERS CAST. RECORD AMBIENT AIR TEMPERATURE AND CONCRETE TEMPERATURE.

**3.0 ANCHOR BOLTS AND EMBEDMENT PLATE**

- A. GENERAL**
- PRODUCTS, SUBMITTALS, EXECUTION, AND TESTING ARE SPECIFIED TO PROVIDE DURABLE ANCHOR BOLTS AND EMBEDMENT PLATES.
- B. SUBMITTALS**
- SUBMIT PRODUCT DATA FOR ANCHORS AND HARDWARE.
  - SUBMIT A 12-INCH LONG PRODUCT SAMPLE OF THE ANCHOR COMPLETE WITH WASHER AND NUT.
  - SUBMIT MILL CERTIFICATES FOR ANCHORS INDICATING YIELD STRENGTH OF ANCHORS.
  - SUBMIT MILL CERTIFICATES FOR THE EMBEDMENT RING INDICATING THAT THE MATERIAL MEETS THE MINIMUM STRENGTH REQUIREMENTS.
  - SUBMIT 5 LABORATORY TENSION TESTS OF ANCHOR COMPLETE WITH THREADS.
  - SUBMIT A TENSIONING CALIBRATION PROCEDURE FOR REVIEW AND APPROVAL, INCLUDING VERIFICATION THAT THE EQUIPMENT PROVIDED AND TENSIONING METHODS USED ARE DELIVERING THE NECESSARY LOCK OFF LOAD.
  - SUBMIT A TENSIONING PROCEDURE FOR REVIEW AND APPROVAL.
  - SUBMIT A TENSION TESTING PROCEDURE FOR REVIEW AND APPROVAL.
  - SUBMIT TENSION TEST DATA FOR ANCHOR BOLTS THAT ARE TESTED INDICATING BOLT LOCATION AND TENSION VALUE.
- C. PRODUCTS**
- ANCHOR BOLTS:** #10 SIZE WITH MATERIAL TO ASTM A615 GRADE 75 KSI, FABRICATED WITH COLD ROLLED THREADS, A MINIMUM YIELD STRENGTH OF 75 KSI, A MINIMUM TENSILE STRENGTH OF 100 KSI, A MAXIMUM THREAD DIAMETER OF 1.375 INCHES, AND A MINIMUM NET AREA OF 1.27 SQUARE INCHES.
  - ANCHOR BOLT SLEEVES:** PVC TO ASTM D1784, MANUFACTURED TO ASTM D2241, SDR21 THICKNESS.
  - EMBEDMENT PLATE:** TO ASTM A36, PLAIN FINISH.
  - HEAVY HEX NUTS:** TO ASTM A194, GRADE 2H, EXCEPT FOR THE DIMENSIONAL REQUIREMENTS. NUTS SHALL BE CAPABLE OF DEVELOPING THE MINIMUM TENSILE STRENGTH OF THE ANCHOR.
  - HARDENED STEEL WASHERS:** TO ASTM F436, PLAIN FINISH.
- D. EXECUTION**
- THE FOLLOWING DIMENSIONAL TOLERANCES SHALL BE ADHERED TO FOR PLACEMENT OF ANCHOR BOLTS:
    - ANCHOR BOLT PLAN LOCATION - PLUS OR MINUS 1/16 INCH.
    - ANCHOR BOLT PLUMBNESS - LESS THAN 1/4 DEGREE.
    - TEMPLATE AND EMBEDMENT RING PLAN DIMENSION - PLUS OR MINUS 3/32 INCH.
    - EMBEDMENT RING LEVEL - PLUS OR MINUS 1/4 INCH.
    - EMBEDMENT RING ELEVATION - PLUS OR MINUS 1/2 INCH.
  - USE A TEMPLATE RING TO SET ANCHOR BOLT PLUMBNESS AND POSITION. ENSURE THE TEMPLATE RING IS SET IN ACCORDANCE WITH THE SPECIFIED CONSTRUCTION TOLERANCES.
  - PLACE AND LEVEL THE EMBEDMENT RING IN ACCORDANCE WITH THE SPECIFIED TOLERANCES. ENSURE THE EMBEDMENT RING IS PROPERLY ANCHORED TO PREVENT MOVEMENT. IT IS ACCEPTABLE TO WELD SUPPLEMENTAL STEEL BRACING TO THE EMBEDMENT RING OR TEMPLATE RING TO PREVENT MOVEMENT.
  - AFTER PLACEMENT OF CONCRETE PEDESTAL, PREVENT WATER FROM ENTERING THE SLEEVE ANNULUS FROM THE TOP SURFACE PRIOR TO SETTING OF TOWER AND GROUTING OF BASEPLATE.
  - AFTER SETTING OF THE LOWER TOWER SECTION(S) AND GROUTING OF THE BASEPLATE AND USING AN APPROVED TENSIONING PROCEDURE, APPLY A LOCK-OFF FORCE TO EACH ANCHOR BOLT WHICH IS GREATER THAN 72 KIPS AND LESS THAN 77 KIPS IN ORDER TO ACHIEVE A MINIMUM TENSION FORCE OF 72 KIPS IN THE ANCHOR. THE LOCK-OFF FORCE SELECTED BY THE CONTRACTOR SHOULD ACCOUNT FOR TENSION LOSSES DUE TO THE TENSIONING PROCEDURE TO ENSURE THE SPECIFIED TENSION TEST VALUE IS ACHIEVED. THE TENSIONING EQUIPMENT FOR THE ANCHOR BOLTS SHOULD BE CALIBRATED IN ACCORDANCE WITH THE APPROVED PROCEDURE ON A REGULAR BASIS.
- E. TESTING AND INSPECTION**
- SUBMIT 5 LABORATORY TENSION TESTS ON PROJECT ANCHORS, COMPLETE WITH THREADS, PERFORMED BY AN INDEPENDENT TESTING LABORATORY. PERFORM TEST IN ACCORDANCE WITH ASTM A370 AND AT A TEMPERATURE OF -20 °F.
  - AFTER ALL BOLTS HAVE BEEN TENSIONED, A MINIMUM OF 10% OF THE TOTAL BOLTS INSTALLED PER FOUNDATION SHALL BE RANDOMLY TESTED TO VERIFY THAT A 72 KIP TENSION LOAD HAS BEEN ACHIEVED BY USE OF AN APPROVED TENSION TESTING PROCEDURE. IF ANY OF THE BOLTS DO NOT MEET THE REQUIRED TENSION TEST VALUE, THEN ALL BOLTS OF THE TOWER MUST BE RETENSIONED AND THE TENSION TEST MUST BE REPEATED. REPEAT THE PROCEDURE UNTIL ALL THE TENSION TESTS PASS.

**- CONTINUED FROM SECTION 4.0**

- C. PRODUCTS**
- CEMENTITIOUS NON-SHRINK GROUT:** PREPACKAGED GROUT CONFORMING TO ASTM C1107, GRADE C WITH A MINIMUM COMPRESSIVE STRENGTH OF 8000 PSI AFTER 28 DAYS.
  - EPOXY NON-SHRINK GROUT:** PREPACKAGED EPOXY GROUT WITH A MINIMUM COMPRESSIVE STRENGTH OF 8000 PSI AFTER 28 DAYS ACCORDING TO ASTM C579 AND A MAXIMUM COEFFICIENT OF THERMAL EXPANSION OF 30 X 10-6 IN/IN/°F IN ACCORDANCE WITH ASTM C531.
- D. EXECUTION**
- MIX, PLACE, AND CURE GROUT IN ACCORDANCE WITH APPROVED MANUFACTURER'S INSTRUCTIONS.
  - FOR CEMENT GROUTS, PROVIDE GROUT SHOULDERS IN ACCORDANCE WITH DRAWING DETAILS. CONTRACTOR HAS THE OPTION TO PROVIDE EITHER A VERTICAL OR 45° SHOULDER. FOR ANY SHOULDER TYPE SELECTED, DO NOT ALLOW GROUT TO BE PLACED AGAINST THE SIDE OF THE TOWER FLANGE.
  - FOR EPOXY GROUTS, POUR GROUT UP THE SIDE OF THE TOWER FLANGE ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS AND ALSO PROVIDE A 1/4" EXPANSION JOINT BETWEEN THE TOWER FLANGE AND THE GROUT.
  - COORDINATE GROUTING PROCEDURES WITH THE REQUIREMENTS OF THE TOWER MANUFACTURER.
- E. TESTING**
- FOR EACH TOWER SET, CAST MINIMUM OF 9 GROUT CUBES, AND PERFORM TWO LABORATORY STRENGTH TESTS AT 28 DAYS (AVERAGE OF THREE CUBE BREAKS FOR EACH STRENGTH TEST) AND IF NECESSARY ONE AT 56 DAYS. CAST ADDITIONAL GROUT CUBES AS REQUIRED TO DETERMINE STRENGTH AT OTHER TIMES.

**5.0 MISCELLANEOUS CONCRETE EMBEDMENTS**

- A. GENERAL**
- COORDINATE THE LOCATION AND PLACEMENT OF GROUNDING GRIDS, CONTROL CONDUIT AND ELECTRICAL CONDUIT.
- B. SUBMITTALS**
- SUBMIT CONDUIT PLACEMENT DETAILS TO THE FOUNDATION ENGINEER FOR APPROVAL SHOWING DISTANCE FROM TOP OF PEDESTAL TO TOP CONDUIT PENETRATION (THROUGH SIDE OF PEDESTAL).
- C. PRODUCTS**
- NO ITEMS.
- D. EXECUTION**
- VERIFY THE LOCATION OF MISCELLANEOUS CONCRETE EMBEDMENTS AND CONDUIT SO AS NOT TO INTERFERE WITH THE FOUNDATION'S STRUCTURAL REINFORCING STEEL.
  - ENSURE THAT MISCELLANEOUS EMBEDMENTS ARE PROPERLY SECURED TO PREVENT MOVEMENT DURING CONCRETE PLACEMENT.
  - TOP OF CONDUIT SHALL BE LOCATED A MINIMUM OF 24 INCHES FROM TOP OF PEDESTAL.

**6.0 SUBMITTALS**

- SUBMIT ONE ELECTRONIC COPY OF THE SUBMITTALS SPECIFIED TO THE FOUNDATION ENGINEER OF RECORD. SUBMITTALS SHALL BE MADE A MINIMUM OF ONE WEEK PRIOR TO ITS INCORPORATION INTO THE WORK.
- THE FOUNDATION ENGINEER OF RECORD WILL REVIEW THE SUBMITTAL FOR CONFORMANCE AND COMPLIANCE WITH THE DRAWINGS AND SPECIFICATIONS.
- THE REVIEW DOES NOT RELIEVE THE CONSTRUCTION CONTRACTOR FROM RESPONSIBILITY FOR ERRORS IN CONSTRUCTION OF THE WORK DUE TO ERRORS CONTAINED IN THOSE DOCUMENTS.

**7.0 TESTING AND INSPECTION REQUIREMENTS**

- SUBMIT A LIST OF THE TESTING COMPANIES THAT WILL BE UTILIZED ON THE PROJECT FOR PERFORMANCE OF TESTS SPECIFIED.
- SUBMIT NAME AND QUALIFICATIONS OF THE GEOTECHNICAL ENGINEER.
- SUBMIT ONE ELECTRONIC COPY OF TESTING AND INSPECTION RECORDS SPECIFIED TO THE FOUNDATION ENGINEER OF RECORD.
- THE FOUNDATION ENGINEER WILL REVIEW THE TESTING AND INSPECTION RECORDS TO CHECK CONFORMANCE WITH THE DRAWINGS AND SPECIFICATIONS.
- EACH FOUNDATION CONSTRUCTED IS SUBJECT TO THE STIPULATED TESTING REQUIREMENTS.

**8.0 TOWER ERECTION AND ANCHOR TENSIONING REQUIREMENTS**

- UP TO THREE TOWER SECTIONS MAY BE ERECTED, LEVELED AND GROUTED WHEN CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED 4000 PSI.
- THE NACELLE AND BLADES MAY BE ERECTED WHEN THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
- ANCHORS MAY BE TENSIONED WHEN THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
- ANCHORS MAY BE TENSIONED WHEN THE GROUT STRENGTH HAS REACHED 5000 PSI.
- THE NACELLE AND BLADES MAY BE ERECTED WHEN THE GROUT STRENGTH HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
- THE NACELLE AND BLADES MAY BE ERECTED UPON COMPLETION OF THE ANCHOR BOLT TENSIONING AND TESTING AS FOUND IN SECTION 3.E.2 VERIFYING THAT THE REQUIRED TENSION VALUE HAS BEEN ACHIEVED.

**FOR CONSTRUCTION**

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**READ THIS DRAWING IN CONJUNCTION WITH DRAWINGS S-01 AND S-03**

A	CAK	-	CAK	11-NOV-09	INITIAL RELEASE
0	CAK	MBJ	CAK	16-DEC-09	FOR CONSTRUCTION
1	CAK	MBJ	CAK	11-JAN-10	REMOVED BOLT V-NOTCH CHARPY TEST. ADDED TEMPERATURE REQUIREMENT.
2	CPB	CAK	CAK	15-FEB-10	REVISED 4.E.1 GROUT TESTING
3	CPB	CAK	CAK	27-MAY-10	REVISED LEAN CONCRETE AND EMBED RING TOLERANCE SPEC.
NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION

CLIENT		11/11/09	12/16/09	1/11/10	2/15/10	5/27/10		
BID								
CONSTRUCTION								
RELEASED TO/FOR	A	0	1	2	3	4	5	
DATE RELEASED								

Project Office:  
**BARR ENGINEERING CO.**  
 4700 WEST 77TH STREET  
 MINNEAPOLIS, MN.  
 55435-4803

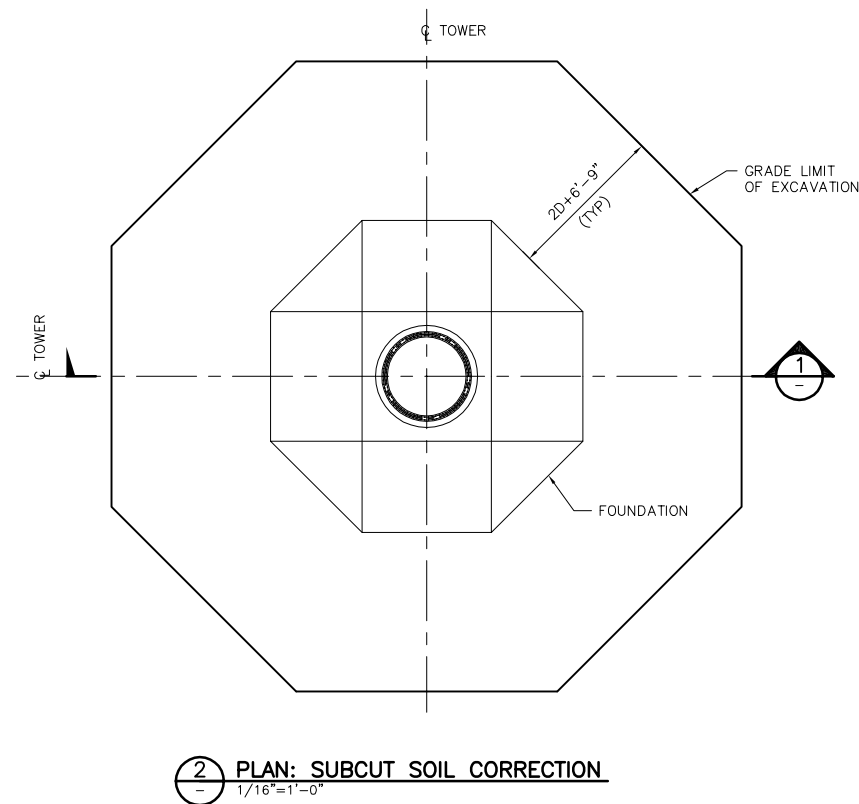
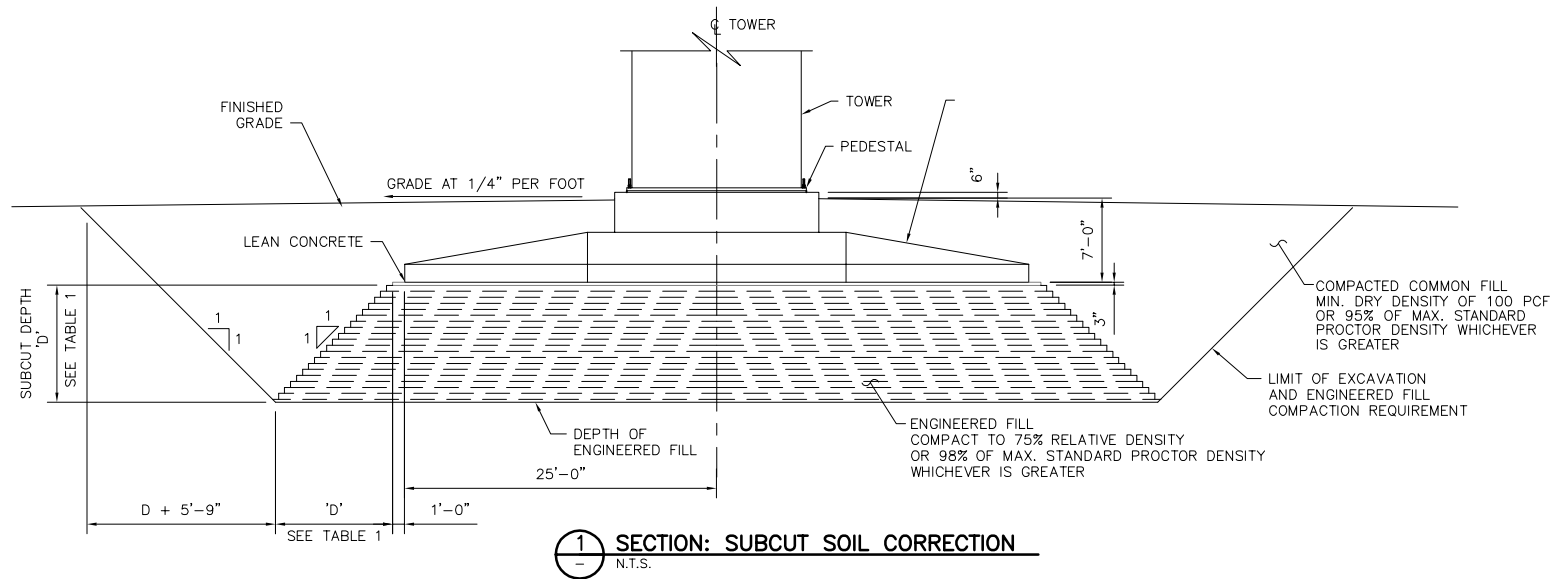
Corporate Headquarters:  
 Minneapolis, Minnesota  
 Ph: 1-800-632-2277  
 Fax: (952) 832-2601  
 www.barr.com

Scale	AS SHOWN
Date	11-NOV-09
Drawn	CAK
Checked	MBJ
Designed	CAK
Approved	CAK

**M.A. MORTENSON COMPANY**  
 MINNEAPOLIS, MINNESOTA

<b>NOBLES WIND PROJECT</b> NOBLES COUNTY, MINNESOTA		BARR PROJECT No. <b>23/53-1002</b>
<b>SPREAD FOOTING FOUNDATION</b> TECHNICAL SPECIFICATIONS AND SUBMITTALS		CLIENT PROJECT No. -
DWG. No.	S-02	REV. No. 3

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OLD TURBINE SITE NO.	NEW TURBINE SITE NO.	DEPTH OF SUBCUT 'D' (FEET)
70	T-070	3
71	T-071	6
77	T-076	8
101	T-109	2

TABLE 1: DEPTH OF SUBCUT 'D'

1.0 SUBGRADE SOIL CORRECTION

A. GENERAL

- ENSURE FOUNDATION SITE IS EXCAVATED, BACKFILLED AND GRADED IN ACCORDANCE WITH SECTION 1 OF THIS DRAWING.
- PERFORM ENGINEERED FILL SUBGRADE CORRECTION AT TURBINE SITES NOTED IN TABLE 1. ALL OTHER SITES DO NOT REQUIRE SOIL CORRECTION.

B. SUBMITTALS

- SUBMIT GRAIN SIZE ANALYSIS, NATURAL MOISTURE CONTENT AND RELATIVE DENSITY TEST DATA FOR ENGINEERED FILL SOIL MATERIALS.
- SUBMIT COMPACTION TEST RESULTS FOR ENGINEERED FILL PLACED BENEATH THE FOUNDATION INDICATING LOCATION OF TEST, DRY DENSITY AND MOISTURE CONTENT OF PLACED ENGINEERED FILL.
- SUBMIT SUBGRADE INSPECTION REPORT FOR EACH FOUNDATION COMPLETED BY A QUALIFIED GEOTECHNICAL ENGINEER.

C. PRODUCTS

- ENGINEERED FILL:** GRANULAR SOIL CONSISTING OF GRAVEL, SAND OR CRUSHED STONE AND COMPLYING WITH MINNESOTA DEPARTMENT OF TRANSPORTATION SPECIFICATION 313B, CLASS 5 AGGREGATE BASE WITH NO GREATER THAN 10% PASSING THE No. 200 SIEVE.

D. EXECUTION

- SCRAPE TOPSOIL FROM THE PLAN AREA AND STORE IN AN OWNER DESIGNATED AREA. THE TOPSOIL WILL BE RE-USED FOR SITE RESTORATION.
- EXCAVATE SOILS TO THE LIMITS SHOWN IN SECTION 1 AND TABLE 1 WHERE APPROVED BY A GEOTECHNICAL ENGINEER, AS NOTED BELOW.
- WHERE NOTED IN TABLE 1, PERFORM SUBGRADE CORRECTION BY SUBCUTTING DEFICIENT SOILS AND BACKFILLING AND COMPACTING WITH ENGINEERED FILL.
- ENGINEERED FILL SOIL CORRECTION IS SUITABLE FOR THE TURBINE SITES LISTED IN TABLE 1.
- FOR EACH TURBINE SITE IDENTIFIED, SUBCUT TO THE DEPTH NOTED IN TABLE 1 AND LIMITS SHOWN IN SECTION 1. HAVE THE PROJECT GEOTECHNICAL ENGINEER VERIFY THE DEPTH OF SUBCUT AT THE TIME OF EXCAVATION AND PREPARE A LETTER REPORT INDICATING THE APPROVED DEPTH OF SUBCUT.
- CONTROL SURFACE WATER OR GROUNDWATER FLOWS INTO THE EXCAVATION USING MEANS DETERMINED BY THE CONTRACTOR. IF SUCH MEANS ARE EMPLOYED, RECORD THE MEANS UNDERTAKEN, SOURCE OF WATER (GROUND OR SURFACE), AND VOLUME OF WATER CONTROLLED. SUBMIT A DEWATERING RECORD TO THE FOUNDATION ENGINEER.
- ENGINEERING FILL, BACKFILL AND COMPACTION.** PLACE AND COMPACT ENGINEERED FILL MATERIALS TO THE LIMITS, DEPTH AND RELATIVE DENSITY INDICATED IN SECTION 1. PLACE AN INITIAL LIFT OF ENGINEERED FILL IMMEDIATELY AFTER COMPLETION OF THE EXCAVATION AND APPROVAL BY THE GEOTECHNICAL ENGINEER. PLACE ENGINEERED FILL IN MAXIMUM LOOSE LIFTS OF 9 INCHES [230 MM] OR LESS TO ACHIEVE THE SPECIFIED RELATIVE DENSITY. PLACE ENGINEERED FILL AT THE DENSITY NOTED ON THE DRAWING.

D. EXECUTION - CONTINUED

- GRADE THE SITE IN ACCORDANCE WITH DRAWINGS TO PREVENT WATER FROM PONDING OVER THE FOUNDATION WHILE MAINTAINING AT LEAST THE MINIMUM DEPTH OF FILL SPECIFIED ON THE DRAWINGS.
  - RESTORE THE SITE IN ACCORDANCE WITH OWNER REQUIREMENTS.
- E. TESTING AND INSPECTION
- FOR EVERY 1000 CUBIC YARDS OF PLACED ENGINEERED FILL,** OBTAIN SAMPLES OF ENGINEERED FILL MATERIALS AND PERFORM GRAIN SIZE ANALYSIS, MOISTURE CONTENT, AND RELATIVE DENSITY TESTS.
  - FOR PLACED AND COMPACTED ENGINEERED FILL PROVIDE TWO DENSITY TESTS PER LIFT INDICATING TEST LOCATION, DRY DENSITY AND MOISTURE CONTENT. IN THE EVENT THAT THE RELATIVE DENSITY REQUIREMENT IS NOT ACHIEVED, RECOMPACT AND RETEST THE ENGINEERED FILL.
  - PROVIDE A SUBGRADE INSPECTION REPORT TO BE COMPLETED BY A GEOTECHNICAL ENGINEER.

READ DRAWING IN CONJUNCTION WITH DRAWINGS S-01 AND S-02

ABBREVIATIONS:

B.O.	BOTTOM OF	O.C.	ON CENTER
C.C.C.	CLEAR CONCRETE COVER	O.D.	OUTSIDE DIAMETER
CL	CENTER LINE	R	RADIUS
EL.	ELEVATION	T&B	TOP AND BOTTOM
E.W.	EACH WAY	T.O.C.	TOP OF CONCRETE
EX.	EXISTING	TYP	TYPICAL
I.D.	INSIDE DIAMETER	UNO	UNLESS NOTED OTHERWISE
MAX.	MAXIMUM	W/	WITH
MIN.	MINIMUM	Ø	DIAMETER
NOM.	NOMINAL		

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NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
1	CAK			11-NOV-09	INITIAL RELEASE
2	CAK			16-DEC-09	FOR CONSTRUCTION
3	CAK			11-JAN-10	CORRECTED FOUNDATION DIMENSION IN SECTION 1
4	MBJ			27-MAY-10	REMOVED GRAVEL RING, LEAN CONCRETE AND COMMON FILL SPECS, UPDATED TABLE 1

I HEREBY CERTIFY THAT THIS PLAN, SPECIFICATION, OR REPORT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY LICENSED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF MINNESOTA.

SIGNATURE: *Christopher A. Kopchynski*

PRINTED NAME: CHRISTOPHER A. KOPCHYNSKI

DATE: 16-DEC-09 REG. NO.: 21869

CLIENT	11/11/09	12/14/09	1/11/10	5/27/10				
BID								
CONSTRUCTION								
RELEASED TO/FOR	A	0	1	2	3	4	5	
DATE RELEASED								

Project Office:  
**BARR ENGINEERING CO.**  
 4700 WEST 77TH STREET  
 MINNEAPOLIS, MN.  
 55435-4803

Corporate Headquarters:  
 Minneapolis, Minnesota  
 Ph: 1-800-632-2277  
 Fax: (952) 832-2601  
 www.barr.com

Scale	AS SHOWN
Date	11-NOV-09
Drawn	CAK
Checked	MBJ
Designed	CAK
Approved	CAK

M.A. MORTENSON COMPANY  
 MINNEAPOLIS, MINNESOTA

NOBLES WIND PROJECT  
 NOBLES COUNTY, MINNESOTA

SPREAD FOOTING FOUNDATION  
 SUBCUT SOIL CORRECTION

BARR PROJECT No.	23/53-1002
CLIENT PROJECT No.	-
DWG. No.	S-03
REV. No.	2

### III. Foundation, Tower and Design Information

#### A. Unit Definitions and Foundation Dimensions

$k \equiv 1000 \cdot \text{lbf}$       meter  $\equiv$  m

Foundation width:	$D := 50 \cdot \text{ft}$	Pedestal diameter:	$C := 17 \cdot \text{ft}$	Average extension of pedestal above ground surface:	$h_{pe} := 6 \text{ in} + \left( \frac{0.25 \text{ in}}{\text{ft}} \right) \cdot \left( \frac{D - C}{4} \right)$
Height of base:	$h_b := 28 \cdot \text{in}$	Height of center (above base):	$h_c := 34 \cdot \text{in}$		$h_{pe} = 8.06 \cdot \text{in}$
Height of soil (from foundation bottom):	$h_s := 7.5 \text{ ft} - h_{pe}$ $h_s = 81.94 \cdot \text{in}$	Height of pedestal:	$h_p := h_s - h_b - h_c + h_{pe}$ $h_p = 28.00 \cdot \text{in}$		



Height of embedment ring above bottom of footing:

$h_c := 10 \cdot \text{in}$

Minimum depth of groundwater below grade:

$d_{GWT} := 3 \cdot \text{ft}$

Minimum depth of groundwater below grade (fatigue):

$d_{GWTf} := 3 \cdot \text{ft}$

Top width:

$B := \frac{D}{1 + \sqrt{2}}$

$B = 20.71 \text{ ft}$

Side width:

$a := \frac{D - B}{2}$

$a = 14.64 \text{ ft}$

Edge slope:

$\xi_1 := \text{atan} \left( \frac{h_c}{a} \right)$

$\xi_1 = 10.95 \cdot \text{deg}$

Corner slope:

$\xi_2 := \text{atan} \left( \frac{2h_c}{B} \right)$

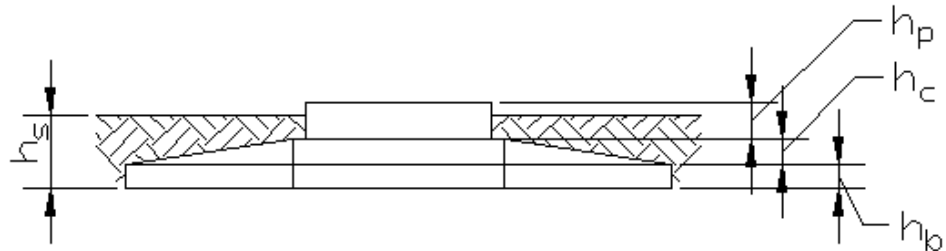
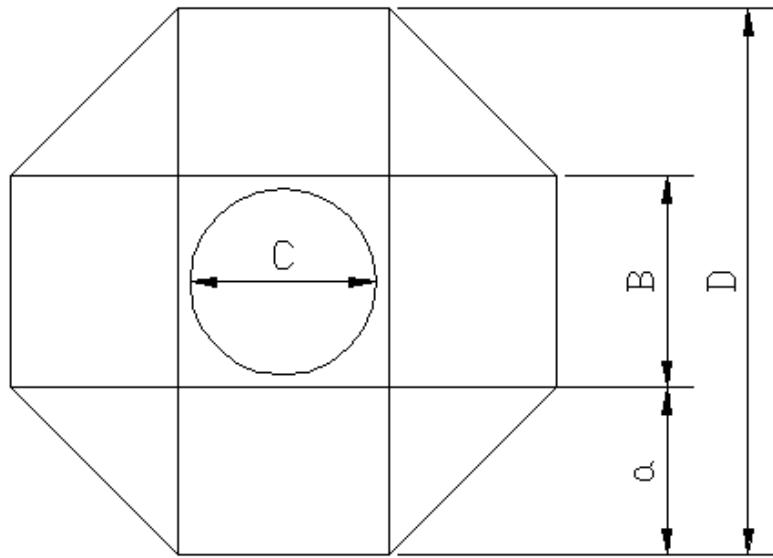
$\xi_2 = 15.30 \cdot \text{deg}$

Bottom cover:

$cc_{bot} := 3 \cdot \text{in}$

Top cover:

$cc_{top} := 2 \cdot \text{in}$



GroundwaterLocation :=  $\left\{ \begin{array}{l} \text{if } d_{GWT} < h_s \\ \quad \left\{ \begin{array}{l} \text{"Within the Base"} \text{ if } d_{GWT} \geq h_s - h_b \\ \text{"Within the Center"} \text{ if } d_{GWT} \geq h_s - h_b - h_c \\ \text{"Within the Pedestal"} \text{ otherwise} \end{array} \right. \\ \text{"Below the Foundation"} \text{ otherwise} \end{array} \right.$

GroundwaterLocation = "Within the Center"

### B. Tower Bottom Flange Dimensions

Flange outside diameter:	$OD := 4556 \cdot \text{mm}$	$OD = 14.95 \text{ ft}$
Flange inside diameter:	$ID := 4000 \cdot \text{mm}$	$ID = 13.12 \text{ ft}$
Total number of bolts (2 circles):	$N := 2 \cdot 70$	
Inner bolt circle diameter:	$D_i := 4130 \cdot \text{mm}$	$D_i = 13.55 \text{ ft}$
Outer bolt circle diameter:	$D_o := 4426 \cdot \text{mm}$	$D_o = 14.52 \text{ ft}$
Average tower diameter:	$d_{\text{tower}} := 4300 \cdot \text{mm} - 26 \text{mm}$	$d_{\text{tower}} = 14.02 \text{ ft}$
Thickness of tower flange:	$t_{\text{flange}} := 75 \cdot \text{mm}$	$t_{\text{flange}} = 2.95 \cdot \text{in}$
Average bolt circle diameter:	$D_{\text{avg}} := \frac{D_i + D_o}{2}$	$D_{\text{avg}} = 168.43 \cdot \text{in}$
Width of flange:	$w_{\text{flange}} := \frac{OD - ID}{2}$	$w_{\text{flange}} = 10.94 \cdot \text{in}$

### C. Stability Safety Factors

Minimum factor of safety:	$FS_{\text{min}} := 1.5$	(Reference 1)
Minimum factor of safety:	$FS_{\text{min}2} := 1.0$	(Reference 1)

### D. Stiffness Requirements

Required dynamic rotational stiffness:	$K_{\psi \text{req}} := \frac{3 \cdot 10^7 \cdot \text{kN} \cdot \text{m}}{\text{rad}}$	(Reference 3)
Required dynamic translational stiffness:	$K_{x \text{req}} := \frac{1.0 \cdot 10^6 \cdot \text{kN}}{\text{m}}$	(Reference 3)

### E. ACI Reinforcing Information

(Reference 1a)

Bar nominal size, diameter (in), area (in <sup>2</sup> ), and weight (lbf/ft):	ACI_bar_table :=	$\left( \begin{array}{cccc} \text{"not used"} & 0.0001 & 0 & 0 \\ 3 & 0.375 & 0.11 & 0.376 \\ 4 & 0.500 & 0.20 & 0.668 \\ 5 & 0.625 & 0.31 & 1.043 \\ 6 & 0.750 & 0.44 & 1.502 \\ 7 & 0.875 & 0.60 & 2.044 \\ 8 & 1.000 & 0.79 & 2.670 \\ 9 & 1.128 & 1.00 & 3.400 \\ 10 & 1.270 & 1.27 & 4.303 \\ 11 & 1.410 & 1.56 & 5.313 \\ 14 & 1.693 & 2.25 & 7.650 \\ 18 & 2.257 & 4.00 & 13.600 \end{array} \right)$
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## F. Material Properties

Friction factor:	$\mu_f := 0.4$	
Concrete strength:	$f_c := 5000 \cdot \text{psi}$	
Steel yield strength (bottom mat):	$f_{yB} := 75000 \text{psi}$	
Steel yield strength (top mat):	$f_{yT} := 75000 \text{psi}$	
Steel verts yield strength:	$f_{yV} := 60000 \cdot \text{psi}$	
Steel modulus of elasticity:	$E_s := 29000 \text{ksi}$	
Density of concrete:	$\gamma_c := 150 \text{pcf}$	
Density of water:	$\gamma_w := 62.4 \text{pcf}$	
Dry density of soil:	$\gamma_d := 105 \text{pcf}$	
Design density of soil above groundwater table:	$\gamma_{sdbot} := \gamma_d \cdot (1 + 15.5\%)$	$\gamma_{sdbot} = 121 \cdot \text{pcf}$
	$\gamma_{sdtop} := \gamma_{sdbot}$	
Specific gravity of soil solids:	$G_s := 2.70$	
Design density of soil below groundwater table:	$\gamma_{ssbot} := \left(1 - \frac{1}{G_s}\right) \cdot \gamma_d + \gamma_w$	$\gamma_{ssbot} = 129 \cdot \text{pcf}$
	$\gamma_{sstop} := \min(120 \text{pcf}, \gamma_{ssbot})$	
Soil wedge angle from vertical:	$\theta := \text{atan}\left(\frac{1}{2}\right)$	$\theta = 26.6 \cdot \text{deg}$
Soil wedge angle from vertical (fatigue):	$\theta_{fat} := \text{atan}\left(\frac{0}{2}\right)$	$\theta_{fat} = 0.00 \cdot \text{deg}$
Concrete modulus of elasticity:	$E_c := 57000 \text{psi} \cdot \sqrt{\frac{f_c}{\text{psi}}}$	$E_c = 4031 \cdot \text{ksi}$
Modulus reduction factor:	$\psi := 0.8$	(Reference 7)
Conversion factor for concrete and grout strength fatigue calculations:	$f_{cck\_conversion} := (1 - 5\%)$	$f_{cck\_conversion} = 0.95$
Modular ratio:	$n_{mod} := \frac{E_s}{\psi \cdot E_c}$	$n_{mod} = 9.0$

## **G. Extreme Loading Conditions**

(Reference 3)

### Misalignment Loading

Base moment:  $M_{align} := 940 \text{ kN}\cdot\text{m}$

$$M_{align} = 693 \cdot \text{ft}\cdot\text{k}$$

Misalignment  
angle (relative to  
 $\Delta := 45 \text{ deg}$

### Normal Extreme

Normal extreme load factor:

$$\alpha_e := 1.35$$

Base moment:

$$M_e := \frac{39515.8}{\alpha_e} \text{ kN}\cdot\text{m}$$

$$M_e = 21589 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_e := \frac{527.6}{\alpha_e} \text{ kN}$$

$$H_e = 88 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{te} := \frac{2762.4}{\alpha_e} \text{ kN}$$

$$W_{te} = 460 \cdot \text{k}$$

### Abnormal Extreme

Abnormal extreme load factor:

$$\alpha_a := 1.10$$

Base moment:

$$M_a := \frac{40528.7 \text{ kN}\cdot\text{m}}{\alpha_a}$$

$$M_a = 27175 \cdot \text{k}\cdot\text{ft}$$

Base shear:

$$H_a := \frac{544.6 \text{ kN}}{\alpha_a}$$

$$H_a = 111 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{ta} := \frac{2234.5 \text{ kN}}{\alpha_a}$$

$$W_{ta} = 457 \cdot \text{k}$$

## **H. Normal Loading Conditions - DLC 1.0**

Base moment:

$$M_N := 17849.4 \cdot \text{kN}\cdot\text{m}$$

$$M_N = 13165 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_N := 254.6 \cdot \text{kN}$$

$$H_N = 57 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_N := 2050.4 \cdot \text{kN}$$

$$W_N = 461 \cdot \text{k}$$

## **I. Normal Loading Conditions - DLC 1.1**

Base moment:

$$M_{1.1} := 17849.4 \cdot \text{kN}\cdot\text{m}$$

$$M_{1.1} = 13165 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_{1.1} := 254.6 \cdot \text{kN}$$

$$H_{1.1} = 57 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{1.1} := 2050.4 \cdot \text{kN}$$

$$W_{1.1} = 461 \cdot \text{k}$$



**J. Earthquake Loading Conditions**

(Reference 1)

Seismic Design Criteria:

Site<sub>Class</sub> := "D"

- Building Occupancy Category: II (Non-Essential Power Facility, Non Hazardous)

- Seismic Design Category: A (Determination of Seismic Design Category)

0.2 Second spectral response:  $S_S := 0.069$

1.0 Second spectral response:  $S_1 := 0.033$

**Table 11.4-1 Site Coefficient,  $F_a$**

**Table 11.4-2 Site Coefficient,  $F_v$**

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				
4	#N/A	#N/A			
	1.6	1.4	1.2	1.1	1

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				
4	#N/A	#N/A			
	2.4	2	1.8	1.6	1.5

$F_a = 1.60$

$F_v = 2.40$

Importance factor:  $I := 1.00$

Adjusted 0.2s response:  $S_{MS} := F_a \cdot S_S$   $S_{MS} = 0.11$

Adjusted 1.0s response:  $S_{M1} := F_v \cdot S_1$   $S_{M1} = 0.08$

Design response:  $S_{DS} := \frac{2}{3} \cdot S_{MS}$   $S_{DS} = 0.07$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$   $S_{D1} = 0.05$

Vertical seismic load effect:  $E_v := \text{if}(S_{DS} \leq 0.125, 0, 0.2 \cdot S_{DS})$   $E_v = 0.00$

Seismic Design Requirements for Nonbuilding/Building Structures:

Response modification coefficient:  $R := 1.5$

Long period transition period:  $T_L := 12$

Period coefficients:  $C_t := 0.02$

$x_{EQ} := 0.75$

Height of structure:  $h_n := 80 \cdot \text{m}$

Approximate fundamental period of structure:  $T_{a1} := C_t \left( \frac{h_n}{\text{ft}} \right)^{x_{EQ}} \cdot \text{s}$   $T_{a1} = 1.3 \text{ s}$

Value of  $C_u$  is calculated based on ASCE 7-05 Table 12.8-1

$C_u = 1.70$

Table 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD	
Design Spectral Response Acceleration Parameter at 1 s, SD1	Coefficient $C_u$
$\geq 0.4$	1.4
0.3	1.4
0.2	1.5
0.15	1.6
$\leq 0.1$	1.7

Maximum approximate period:  $T_{a2} := C_u \cdot T_{a1}$   $T_{a2} = 2.22 \text{ s}$

Design period:  $T := \min(T_{a1}, T_{a2})$   $T = 1.3 \text{ s}$

Seismic response coefficients:  $C_{s1} := \frac{S_{DS}}{\frac{R}{I}}$   $C_{s1} = 0.049$

$$C_{s2} := \text{if} \left[ \frac{T}{\text{sec}} \leq T_L, \frac{S_{D1}}{\left( \frac{T}{\text{sec}} \right) \cdot \left( \frac{R}{I} \right)}, \frac{S_{D1} \cdot T_L}{\left( \frac{T}{\text{sec}} \right)^2 \cdot \left( \frac{R}{I} \right)} \right] = 0.027$$

Non-Building structure:  $C_{s3} := 0.03$

$$C_{s4} := \text{if} \left( S_1 > 0.6, \frac{0.8 \cdot S_1}{\frac{R}{I}}, 0 \right) \quad C_{s4} = 0.00$$

$C_s := \max(\min(C_{s1}, C_{s2}), C_{s3}, C_{s4})$   $C_s = 0.03$

Minimum base shear:  $V := C_s \cdot W_{1.1}$   $V = 14 \cdot \text{kip}$

Structure Weights and Centers of Gravity:

(Reference 3)

Total weight of structure:

$$W_{1.1} = 461 \cdot \text{kip}$$

Weight of Component

Approximate Center of Gravity

Tower bottom section::

$$W_1 := 121500 \text{ lbf}$$

$$h_1 := \frac{21925 \text{ mm}}{2}$$

Tower lower mid section:

$$W_2 := 112240 \text{ lbf}$$

$$h_2 := \frac{25912 \text{ mm}}{2} + 21925 \text{ mm}$$

Tower upper mid section:

$$W_3 := 104090 \text{ lbf}$$

$$h_3 := \frac{29463 \text{ mm}}{2} + 25912 \text{ mm} + 21925 \text{ mm}$$

Tower top section:

$$W_4 := 0 \cdot \text{lbf}$$

$$h_4 := 0 \cdot \text{mm}$$

Wind turbine nacelle and rotor:

$$h_5 := 80 \cdot \text{m}$$

Total tower weight:

$$W_{\text{tower}} := W_1 + W_2 + W_3 + W_4$$

$$W_{\text{tower}} = 338 \cdot \text{kip}$$

Wind turbine nacelle and rotor:

$$W_5 := W_{1.1} - W_{\text{tower}}$$

$$W_5 = 123 \cdot \text{kip}$$

Summary:

$$W = \begin{pmatrix} 0 \\ 122 \\ 112 \\ 104 \\ 0 \\ 123 \end{pmatrix} \cdot \text{kip}$$

$$h = \begin{pmatrix} 0.0 \\ 11.0 \\ 34.9 \\ 62.6 \\ 0.0 \\ 80.0 \end{pmatrix} \cdot \text{m}$$

Determine Design Base Shear:

Exponent related to period of structure:

$$k_0 := \text{if} \left[ \frac{T}{\text{sec}} \leq 0.5, 1, \text{if} \left[ \frac{T}{\text{sec}} \geq 2.5, 2, 2 - \left( \frac{2.5 - \frac{T}{\text{sec}}}{2} \right) \right] \right]$$

$$k_0 = 1.4$$

Number of tower components:

$$\text{ntc} := 1 \dots 5$$

Vertical Distribution Factor:

$$C_{v_{\text{ntc}}} := \frac{W_{\text{ntc}} \cdot \left( \frac{h_{\text{ntc}}}{\text{ft}} \right)^{k_0}}{\sum_{i=1}^5 \left[ W_i \cdot \left( \frac{h_i}{\text{ft}} \right)^{k_0} \right]}$$

Lateral Seismic Force at Each Level:

$$F_{\text{ntc}} := C_{v_{\text{ntc}}} \cdot V$$

$C_{v_{\text{ntc}}} =$

0.031
0.146
0.308
0.000
0.514

$F_{\text{ntc}} =$

0.4
2.0
4.3
0.0
7.1

·kip

Total lateral seismic force:

$$H_{\text{EQ}} := \sum_{i=1}^5 (F_i)$$

$$H_{\text{EQ}} = 14 \cdot \text{kip}$$

Earthquake overturning moment:

$$M_{\text{EQ}} := \sum_{i=1}^5 (F_i \cdot h_i)$$

$$M_{\text{EQ}} = 2988 \cdot \text{ft} \cdot \text{kip}$$

Base moment:

$$M_{\text{OE}} := \sqrt{M_{1.1}^2 + M_{\text{EQ}}^2}$$

$$M_{\text{OE}} = 13500 \cdot \text{ft} \cdot \text{k}$$

Base shear:

$$H_{\text{OE}} := \sqrt{H_{1.1}^2 + H_{\text{EQ}}^2}$$

$$H_{\text{OE}} = 59 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{\text{OE}} := W_{1.1}$$

$$W_{\text{OE}} = 461 \cdot \text{k}$$

### K. Fatigue Loading Conditions

(Reference 3)

Mean shear:

$$H_{\text{mean}} := \sqrt{212.6^2 + 3.4^2} \text{ kN}$$

$$H_{\text{mean}} = 213 \cdot \text{kN}$$

Mean overturning moment:

$$M_{\text{mean}} := \sqrt{15890^2 + 1310^2} \text{ kN}\cdot\text{m}$$

$$M_{\text{mean}} = 15944 \cdot \text{kN}\cdot\text{m}$$

Turbine & tower mean weight:

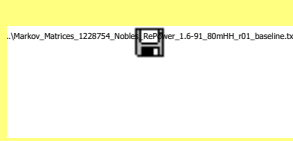
$$W_{\text{mean}} := 1920.3 \text{ kN}$$

$$W_{\text{mean}} = 432 \cdot \text{kip}$$

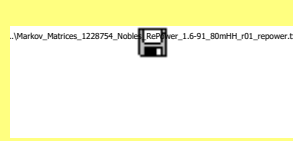
Fatigue Loading (Markov Matrix):

input raw Markov Matrix data - width/range, mean, cycles

baseline :=



repower :=



$$\text{rows}(\text{repower}) = 41696$$

$$\text{rows}(\text{baseline}) + \text{rows}(\text{repower}) = 81691$$

$$\text{markov\_baseline} := \text{augment}(\text{baseline}^{\langle 0 \rangle}, \text{baseline}^{\langle 1 \rangle}, \text{baseline}^{\langle 2 \rangle})$$

$$\text{markov\_repower} := \text{augment}(\text{repower}^{\langle 0 \rangle}, \text{repower}^{\langle 1 \rangle}, \text{repower}^{\langle 2 \rangle})$$

Years of operation on baseline turbine:

$$\text{Year}_{\text{baseline}} := 11$$

Years of fatigue assumed baseline turbine:

$$\text{Year}_{\text{fat\_baseline}} := 11$$

$$N_{\text{fat\_baseline}} := \left( \frac{\text{Year}_{\text{baseline}}}{\text{Year}_{\text{fat\_baseline}}} \right) \cdot \text{baseline}^{\langle 2 \rangle}$$

$$N_{\text{fat\_baseline}_{50}} = 424000$$

Years of operation on repower turbine:

$$\text{Year}_{\text{repower}} := 20$$

Years of fatigue assumed repower turbine:

$$\text{Year}_{\text{fat\_repower}} := 20$$

$$N_{\text{fat\_repower}} := \left( \frac{\text{Year}_{\text{repower}}}{\text{Year}_{\text{fat\_repower}}} \right) \cdot \text{repower}^{\langle 2 \rangle}$$

$$N_{\text{fat\_repower}_{105}} = 490000$$

$$N_{\text{fat}} := \text{stack}(N_{\text{fat\_baseline}}, N_{\text{fat\_repower}}) \quad \text{rows}(N_{\text{fat}}) = 81691$$

$$N_{\text{fat}_{509}} = 417000$$

Minimum Bins

$$\text{MinBin\_baseline} := \text{baseline}^{\langle 1 \rangle} - \frac{\text{baseline}^{\langle 0 \rangle}}{2}$$

$$\text{MinBin\_repower} := \text{repower}^{\langle 1 \rangle} - \frac{\text{repower}^{\langle 0 \rangle}}{2}$$

$$\text{MinBin\_total} := \text{stack}(\text{MinBin\_baseline}, \text{MinBin\_repower})$$

$$\text{rows}(\text{MinBin\_total}) = 81691$$

Maximum Bins

$$\text{MaxBin\_baseline} := \text{baseline}^{\langle 1 \rangle} + \frac{\text{baseline}^{\langle 0 \rangle}}{2}$$

$$\text{MaxBin\_repower} := \text{repower}^{\langle 1 \rangle} + \frac{\text{repower}^{\langle 0 \rangle}}{2}$$

$$\text{MaxBin\_total} := \text{stack}(\text{MaxBin\_baseline}, \text{MaxBin\_repower})$$

$$\text{rows}(\text{MaxBin\_total}) = 81691$$

Bin counters:

$$qt := \text{length}(N_{fat}) - 1 = 81690$$

$$qr := 0, 1 .. qt$$

North Bins

$$\text{MinBin}_{qr} := \text{if}(\text{MinBin}_{total_{qr}} < 0, 0, \text{MinBin}_{total_{qr}})$$

$$\text{MaxBin}_{qr} := \text{if}(\text{MaxBin}_{total_{qr}} < 0, 0, \text{MaxBin}_{total_{qr}})$$

Minimum Moment (for Miner's Rule):

$$M_{fatminnorth_{qr}} := \text{MinBin}_{qr} \cdot \text{kN} \cdot \text{m}$$

$$H_{minnorth_{qr}} := \max\left(\frac{M_{fatminnorth_{qr}}}{M_{mean}} \cdot H_{mean}, 0.01 \text{ kN}\right)$$

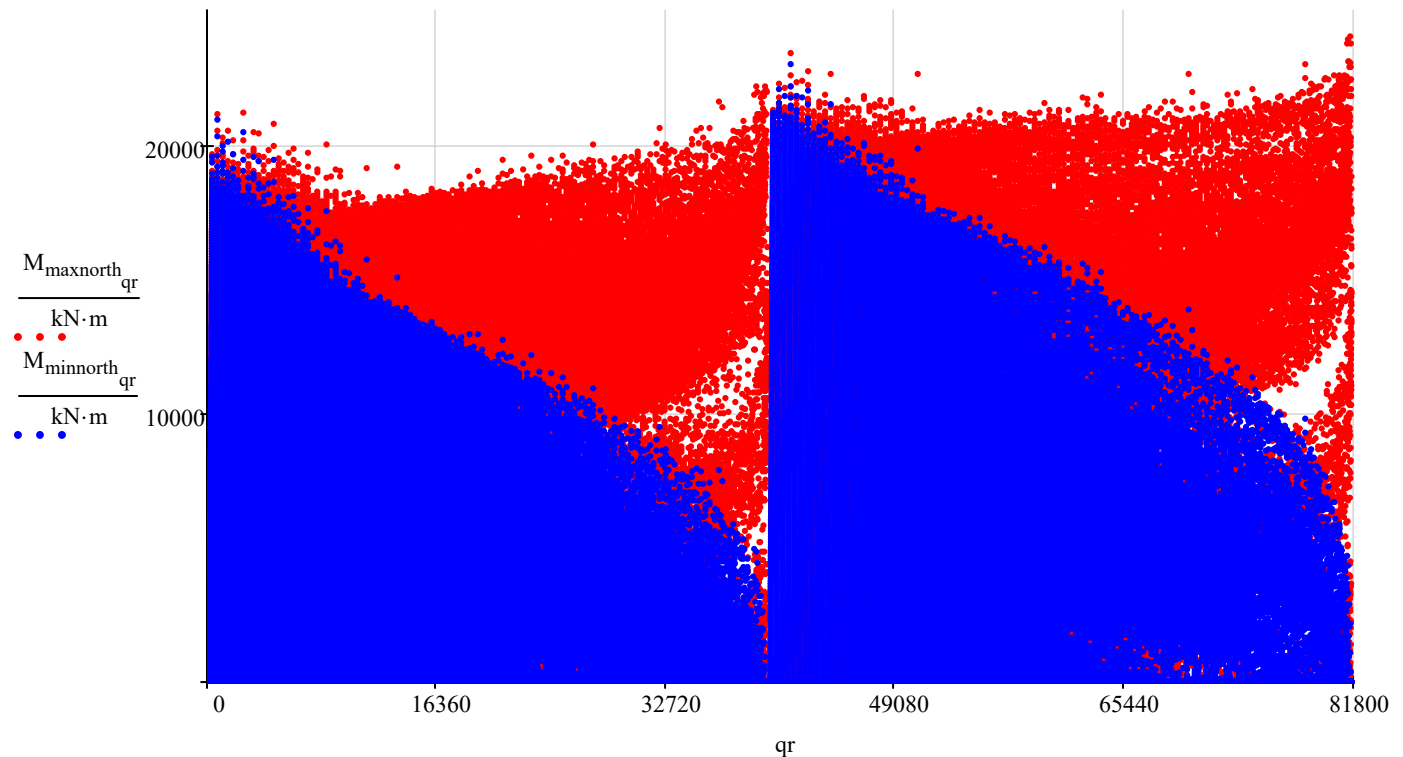
$$M_{minnorth_{qr}} := M_{fatminnorth_{qr}} + H_{minnorth_{qr}} \cdot (h_p + h_c + h_b)$$

Maximum Moments:

$$M_{maxfatigue} := \max(M_{maxnorth})$$

$$M_{minfatigue} := \max(M_{minnorth})$$

Fatigue moments are plotted below:



$$\text{Doubleqr} := 0, 1 .. 2 \cdot qt + 1$$

South Bins

$$\text{MinBin}_{qr} := \text{if}(\text{MaxBin}_{total_{qr}} > 0, 0, -1 \cdot \text{MaxBin}_{total_{qr}})$$

$$\text{MaxBin}_{qr} := \text{if}(\text{MinBin}_{total_{qr}} > 0, 0, -1 \cdot \text{MinBin}_{total_{qr}})$$

Maximum Moment (for Miner's Rule):

$$M_{fatmaxnorth_{qr}} := \text{MaxBin}_{qr} \cdot \text{kN} \cdot \text{m}$$

$$H_{maxnorth_{qr}} := \max\left(\frac{M_{fatmaxnorth_{qr}}}{M_{mean}} \cdot H_{mean}, 0.01 \text{ kN}\right)$$

$$M_{maxnorth_{qr}} := M_{fatmaxnorth_{qr}} + H_{maxnorth_{qr}} \cdot (h_p + h_c + h_b)$$

$$M_{maxfatigue} = 24010 \cdot \text{kN} \cdot \text{m}$$

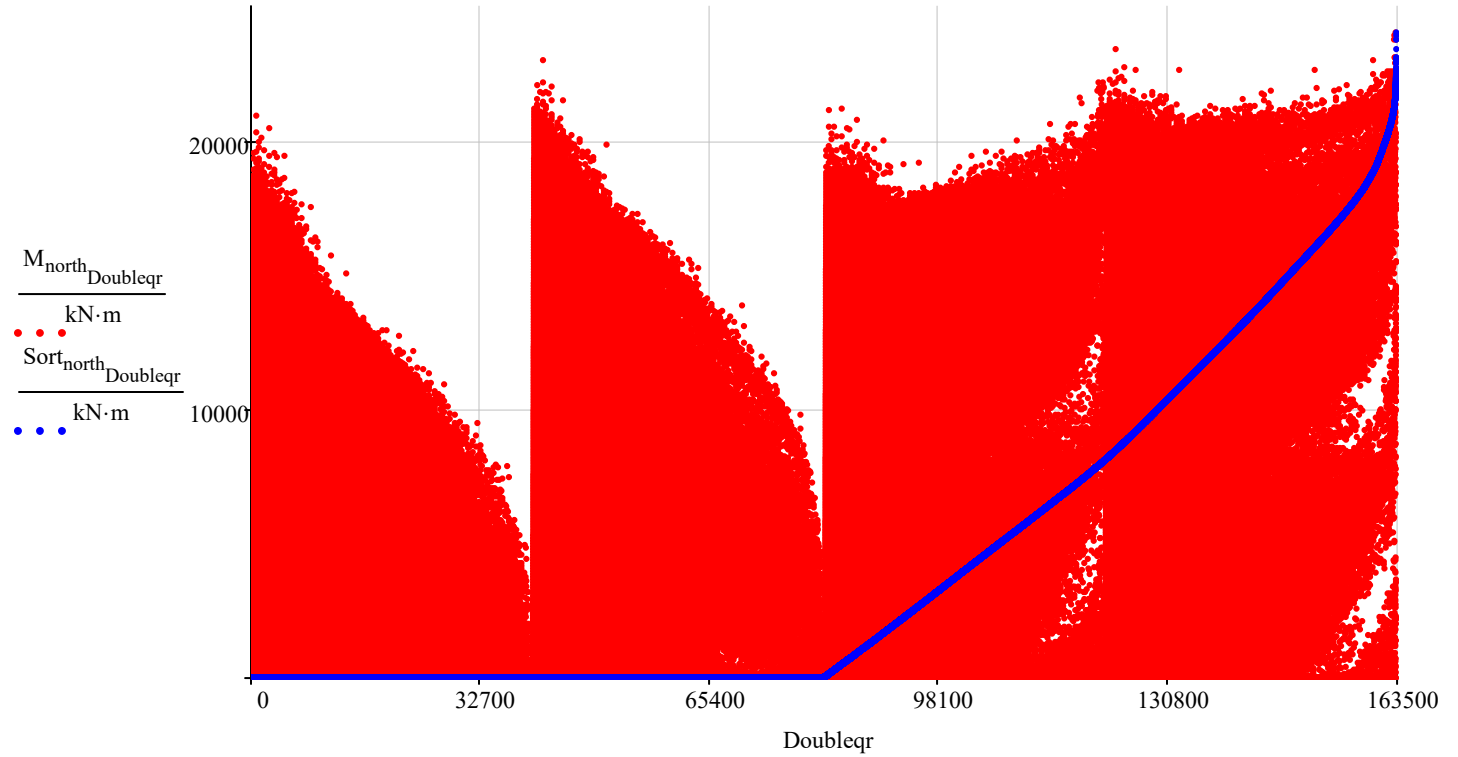
$$M_{minfatigue} = 22980 \cdot \text{kN} \cdot \text{m}$$

### L. Stack Min and Max Fatigue Matrices and Sort

Stack min and max  
 fatigue matrices  
 and sort in  
 ascending order:

$$M_{\text{north}_{\text{Doubleqr}}} := \begin{cases} M_{\text{minnorth}_{\text{Doubleqr}}} & \text{if } \text{Doubleqr} \leq qt \\ M_{\text{maxnorth}_{\text{Doubleqr}-qt-1}} & \text{if } \text{Doubleqr} > qt \end{cases}$$

$$\text{Sort}_{\text{north}} := \text{sort}(M_{\text{north}})$$



### M. Develop Unique Fatigue Matrix and Parse Zero Values

Develop matrix containing only the unique fatigue values:

$$\text{Unique}_{\text{north}_{\text{Doubleqr}}} := \begin{cases} \text{Sort}_{\text{north}_0} & \text{if Doubleqr} = 0 \\ \text{Sort}_{\text{north}_{\text{Doubleqr}}} & \text{if Doubleqr} \neq 0 \wedge \text{Sort}_{\text{north}_{\text{Doubleqr}}} \neq \text{Sort}_{\text{north}_{\text{Doubleqr}-1}} \\ 0 & \text{if Doubleqr} \neq 0 \wedge \text{Sort}_{\text{north}_{\text{Doubleqr}}} = \text{Sort}_{\text{north}_{\text{Doubleqr}-1}} \end{cases}$$

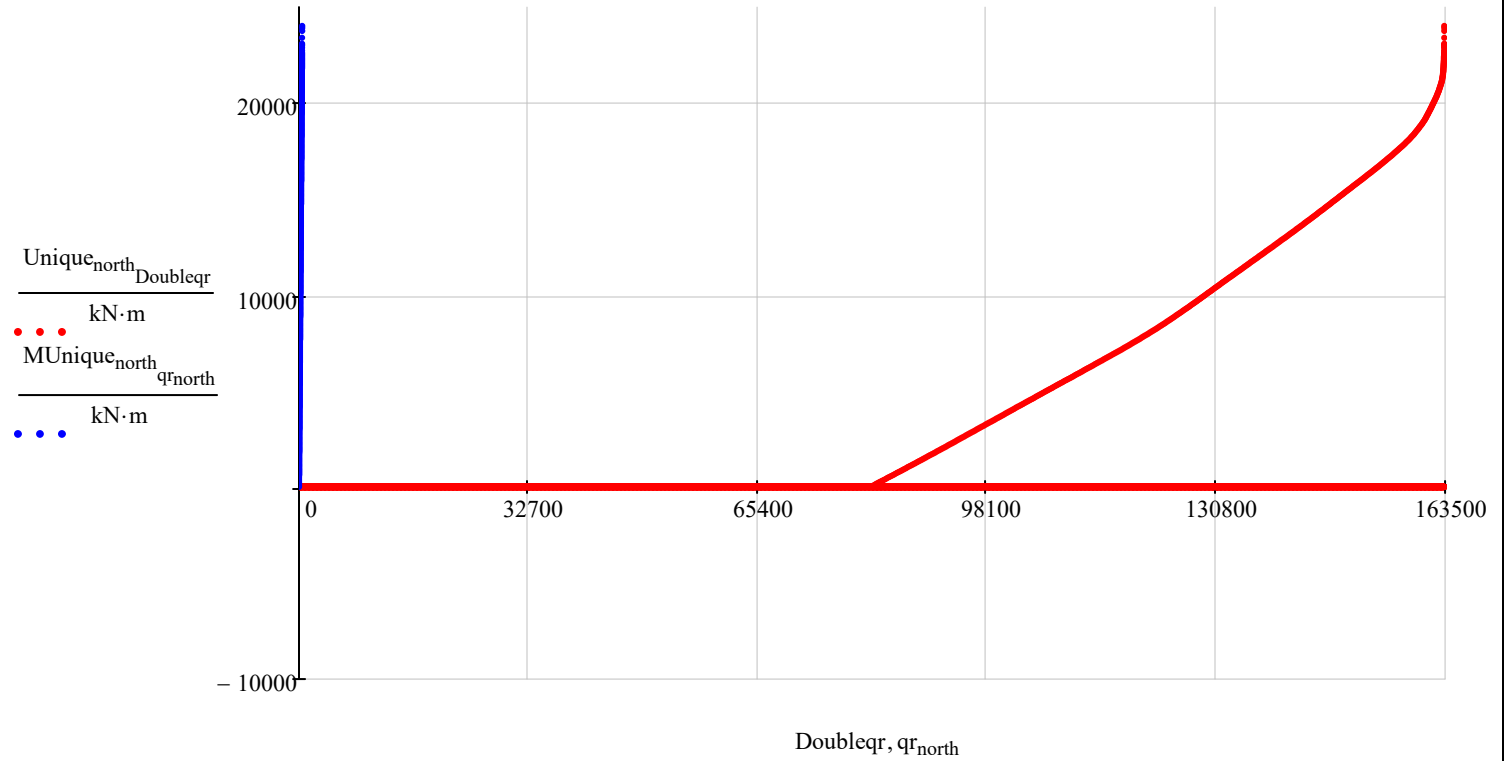
Remove all "zero" entries leaving only "non-zero" entries in the MUnique matrix:

$$\text{MUnique}_{\text{north}} := \begin{cases} \text{trim}(\text{Unique}_{\text{north}}, \text{match}(0, \text{Unique}_{\text{north}})) & \text{if } \min(\text{Unique}_{\text{north}}) = 0 \\ \text{Unique}_{\text{north}} & \text{if } \min(\text{Unique}_{\text{north}}) > 0 \end{cases}$$

Determine the quantity of "non-zero" entries in the MUnique matrix to use as counters:

$$\text{qt}_{\text{north}} := \text{length}(\text{MUnique}_{\text{north}}) - 1 = 448$$

$$\text{qr}_{\text{north}} := 0, 1 \dots \text{qt}_{\text{north}}$$





## IV. Stability Analysis and Bearing Length Evaluation

### A. Foundation Volume and Weight Calculations

Foundation plan area:

$$A := D^2 - 2 \cdot \left( \frac{D - B}{2} \right)^2 \quad A = 2071 \cdot \text{ft}^2$$

Volume of pedestal:

$$v_p := \frac{\pi \cdot C^2}{4} \cdot h_p \quad v_p = 20 \cdot \text{yd}^3$$

Weight of pedestal:

$$W_p := v_p \cdot \gamma_c \quad W_p = 79 \cdot \text{k}$$

Volume of footing:

$$v_f := A \cdot h_b + B^2 \cdot h_c + 4 \cdot \left( \frac{1}{2} \cdot \frac{1}{3} \cdot h_c \cdot a^2 + \frac{1}{2} \cdot B \cdot h_c \cdot a \right) \quad v_f = 303 \cdot \text{yd}^3$$

Weight of footing:

$$W_f := v_f \cdot \gamma_c \quad W_f = 1226 \cdot \text{k}$$

Total volume of concrete:

$$v_c := v_f + v_p \quad v_c = 322 \cdot \text{yd}^3$$

Total volume of soil:

$$v_s := A \cdot h_s - v_f - \frac{\pi \cdot C^2}{4} \cdot (h_s - h_b - h_c) \dots + \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot (h_s - h_b)^2 \quad v_s = 238 \cdot \text{yd}^3$$

Total volume of soil (fatigue)

$$v_{\text{sfat}} := A \cdot h_s - v_f - \frac{\pi \cdot C^2}{4} \cdot (h_s - h_b - h_c) \dots + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot (h_s - h_b)^2 \quad v_{\text{sfat}} = 207 \cdot \text{yd}^3$$

Equivalent buoyant force:

$$F_b := \gamma_w \cdot \begin{cases} \text{if } d_{\text{GWT}} < h_s \\ \left| \begin{array}{l} A \cdot (h_s - d_{\text{GWT}}) \text{ if } d_{\text{GWT}} \geq h_s - h_b \\ A \cdot (h_s - d_{\text{GWT}}) + \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot (h_s - h_b - d_{\text{GWT}})^2 \text{ otherwise} \end{array} \right. \\ 0 \text{ otherwise} \end{cases} \quad F_b = 501 \cdot \text{kip}$$

Equivalent buoyant force (fatigue):

$$F_{\text{bfat}} := \gamma_w \cdot \begin{cases} \text{if } d_{\text{GWTF}} < h_s \\ \left| \begin{array}{l} A \cdot (h_s - d_{\text{GWTF}}) \text{ if } d_{\text{GWTF}} \geq h_s - h_b \\ A \cdot (h_s - d_{\text{GWTF}}) + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \text{ otherwise} \end{array} \right. \\ 0 \text{ otherwise} \end{cases} \quad F_{\text{bfat}} = 495 \cdot \text{kip}$$

Total weight of soil:

$$W_s := \begin{cases} \text{if } d_{GWT} < h_s - h_b \\ \left[ \begin{aligned} &\gamma_{sdbot} \cdot d_{GWT} \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) + \gamma_{ssbot} \cdot (h_s - h_b - h_c - d_{GWT}) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots \\ &+ \gamma_{ssbot} \cdot [A \cdot (h_b + h_c) - v_f] \dots \\ &+ \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ssbot} (h_s - h_b - d_{GWT})^2 + \gamma_{sdbot} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] \\ &\gamma_{sdbot} \cdot (h_s - h_b - h_c) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots \\ &+ \gamma_{sdbot} \cdot \int_{h_s - h_b - h_c}^{d_{GWT}} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \dots \\ &+ \gamma_{ssbot} \cdot \left[ \int_{d_{GWT}}^{h_s - h_b} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \right] \dots \\ &+ \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ssbot} (h_s - h_b - d_{GWT})^2 + \gamma_{sdbot} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] \end{aligned} \right] \\ \gamma_{ssbot} \cdot \gamma_{sdbot} \text{ otherwise} \end{cases} \quad \begin{matrix} \text{if } d_{GWT} < h_s - h_b - h_c \\ \\ \\ \\ \text{otherwise} \end{matrix}$$

$$W_s = 786 \cdot k$$

Total weight of soil (fatigue):

$$W_{\text{sfat}} := \begin{cases} \text{if } d_{\text{GWTF}} < h_s - h_b \\ \gamma_{\text{sdbot}} \cdot d_{\text{GWTF}} \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) + \gamma_{\text{ssbot}} \cdot (h_s - h_b - h_c - d_{\text{GWTF}}) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots & \text{if } d_{\text{GWTF}} < h_s - h_b - h_c \\ + \gamma_{\text{ssbot}} \cdot \left[ A \cdot (h_b + h_c) - v_f \right] \dots \\ + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot \left[ \gamma_{\text{ssbot}} (h_s - h_b - d_{\text{GWTF}})^2 + \gamma_{\text{sdbot}} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] \\ \\ \gamma_{\text{sdbot}} \cdot (h_s - h_b - h_c) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots & \text{otherwise} \\ + \gamma_{\text{sdbot}} \cdot \int_{h_s - h_b - h_c}^{d_{\text{GWTF}}} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \dots \\ + \gamma_{\text{ssbot}} \cdot \left[ \int_{d_{\text{GWTF}}}^{h_s - h_b} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \dots \right. \\ \left. + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot \left[ \gamma_{\text{ssbot}} (h_s - h_b - d_{\text{GWTF}})^2 + \gamma_{\text{sdbot}} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] \right] \\ v_{\text{sfat}} \cdot \gamma_{\text{sdbot}} \text{ otherwise} \end{cases}$$

$$W_{\text{sfat}} = 683744 \cdot \text{lbf}$$

Total dead weight seismic load:  $W_{\text{EQ}} := W_p + W_f + W_{\text{OE}} + W_s - F_b$   $W_{\text{EQ}} = 2052 \cdot \text{k}$

Total dead weight fatigue load:  $W_{\text{fat}} := W_p + W_f + W_{\text{mean}} + W_{\text{sfat}} - F_{\text{bfat}}$   $W_{\text{fat}} = 1926 \cdot \text{k}$

## B. Stability Calculations - Extreme Loading

Determine controlling extreme load case:

$$\text{if}(M_e > M_a, \text{"Normal Controls"}, \text{"Abnormal Controls"}) = \text{"Abnormal Controls"}$$

Extreme Tower Base Moment:

$$M := \sqrt{(\text{if}(M_e > M_a, M_e, M_a) + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} = 27670 \cdot \text{k} \cdot \text{ft}$$

Extreme Tower Base Shear:

$$H := \text{if}(M_e > M_a, H_e, H_a) \quad H = 111 \cdot \text{k}$$

Extreme Tower Weight:

$$W_t := \text{if}(M_e > M_a, W_{te}, W_{ta}) \quad W_t = 457 \cdot \text{k}$$

Total dead weight wind load:

$$W_W := W_p + W_f + W_t + W_s - F_b \quad W_W = 2047 \cdot \text{k}$$

Overturning wind moment:

$$M_{oW} := M + (h_b + h_c + h_p) \cdot H \quad M_{oW} = 28504 \cdot \text{k} \cdot \text{ft}$$

Wind load friction resistance at base:

$$H_{frW} := \mu_f \cdot (W_W) \quad H_{frW} = 819 \cdot \text{k}$$

Factor of safety against sliding:

$$FS_{sW} := \frac{H_{frW}}{H} \quad FS_{sW} = 7.36$$

Seismic load friction resistance at base:

$$H_{frEQ} := \mu_f \cdot (W_{EQ}) \cdot (1 - E_v) \quad H_{frEQ} = 821 \cdot \text{k}$$

Factor of safety against sliding:

$$FS_{sEQ} := \frac{H_{frEQ}}{H_{OE}} \quad FS_{sEQ} = 13.94$$

Determine controlling load case:

$$FS_s := \min(FS_{sW}, FS_{sEQ}) \quad FS_s = 7.36$$

$$\text{if}(FS_s \geq FS_{\min}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$$

Resisting moment:

$$M_{rW} := W_W \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right) \quad M_{rW} = 51181 \cdot \text{ft} \cdot \text{k}$$

Factor of safety against overturning:

$$FS_{oW} := \frac{M_{rW}}{M_{oW}} \quad FS_{oW} = 1.80$$

Overturning seismic moment:

$$M_{oEQ} := \sqrt{(M_{OE} + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_{OE} = 14440 \cdot \text{k} \cdot \text{ft}$$

Resisting moment:

$$M_{rEQ} := (1 - E_v) \cdot W_{EQ} \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right) \quad M_{rEQ} = 51288 \cdot \text{ft} \cdot \text{k}$$

Factor of safety against overturning:

$$FS_{oEQ} := \frac{M_{rEQ}}{M_{oEQ}} \quad FS_{oEQ} = 3.55$$

Minimum factor of safety:

$$FS_{\min} = 1.50$$

Determine controlling load case:

$$FS_o := \min(FS_{oW}, FS_{oEQ}) \quad FS_o = 1.80$$

$$\text{if}(FS_o \geq FS_{\min}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$$

Resisting moment (reduced):  $M_{rW\_red} := 0.6W_W \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$   $M_{rW\_red} = 30709 \cdot \text{ft} \cdot \text{k}$

Factor of safety against overturning (alternate):  $FS_{oW\_alt} := \frac{M_{rW\_red}}{M_{oW}}$   $FS_{oW\_alt} = 1.08$

Overturning seismic moment (reduced):  $M_{oEQ\_alt} := 0.7 \cdot M_{oEQ}$   $M_{oEQ\_alt} = 10108 \cdot \text{k} \cdot \text{ft}$

Resisting moment (reduced):  $M_{rEQ\_red} := 0.6(1 - E_v) \cdot W_{EQ} \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$   $M_{rEQ\_red} = 30773 \cdot \text{ft} \cdot \text{k}$

Factor of safety against overturning (alternate):  $FS_{oEQ\_alt} := \frac{M_{rEQ\_red}}{M_{oEQ\_alt}}$   $FS_{oEQ\_alt} = 3.04$

Minimum factor of safety:  $FS_{min2} = 1.00$

Determine controlling load case:  $FS_{o2} := \min(FS_{oW\_alt}, FS_{oEQ\_alt})$   $FS_{o2} = 1.08$

$\text{if}(FS_{o2} \geq FS_{min2}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$

### C. Soil Pressure Calculations - Extreme Loading

Side length of square inscribed inside of foundation octagon:  $S_f := \sqrt{2a^2 + 2 \cdot a \cdot B + B^2}$   $S_f = 38.3 \text{ ft}$

"Major" width of octagon along widest section:  $H_f := \sqrt{B^2 + D^2}$   $H_f = 54.1 \text{ ft}$

Distance from widest section of octagon to edge of square:  $W_e := \frac{H_f - S_f}{2}$   $W_e = 7.93 \text{ ft}$

Moment of inertia for octagon about any axis through centroid:  $I_{fdn} := \frac{8 \cdot B^4}{192} \cdot \cot\left(\frac{2 \cdot \pi}{16}\right) \left(3 \cot\left(\frac{2 \cdot \pi}{16}\right)^2 + 1\right)$   $I_{fdn} = 342112 \cdot \text{ft}^4$

Section modulus of foundation for normal orientation:  $S_{normal} := \frac{2I_{fdn}}{D}$   $S_{normal} = 13684 \cdot \text{ft}^3$

Section modulus of foundation for orientation rotated by 22.5 degrees:  $S_{rotated} := \frac{2 \cdot I_{fdn}}{H_f}$   $S_{rotated} = 12643 \cdot \text{ft}^3$

$\text{if}(S_{normal} > S_{rotated}, \text{"Rotated Controls"}, \text{"Normal Controls"}) = \text{"Rotated Controls"}$

### 1) Assumed Soil Pressure Case 1

Assume triangular distribution with length of bearing ( $L_b$ ) greater than  $W_e$  but less than or equal to half the major octagon width ( $H_f/2$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2047 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 26894 \cdot ft \cdot k$$

Guess:  $L := \frac{2W_e + H_f}{4} \quad f_{max} := 5000 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_4 = 30.2 \text{ ft} \quad f_4 = 4173 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil Bearing Length:  $L_{b4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, L_4, 0 \cdot ft \right) \quad L_{b4} = 0.0 \cdot ft$

Maximum Soil Bearing Pressure:  $f_{max4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, f_4, 0 \cdot psf \right) \quad f_{max4} = 0 \cdot psf$

## 2) Assumed Soil Pressure Case 2

Assume triangular distribution with length of bearing ( $L_b$ ) greater than half the major octagon width ( $H_f/2$ ) but less than or equal to difference between the full octagon width ( $H_f$ ) and  $W_e$ .

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2047 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 26894 \cdot ft \cdot k$$

Guess:  $L := \frac{3H_f - 2W_e}{4}$   $f_{max} := 5000 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_5 = 30.2 \text{ ft} \quad f_5 = 4171 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, L_5, 0 \cdot ft \right)$   $L_{b5} = 30.2 \cdot ft$

Maximum soil bearing pressure:  $f_{max5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, f_5, 0 \cdot psf \right)$   $f_{max5} = 4171 \cdot psf$

### 3) Assumed Soil Pressure Case 3

Assume triangular distribution with length of bearing ( $L_b$ ) greater than the difference between the full octagon width ( $H_f$ ) and  $W_e$  but less than the full octagon width ( $H_f$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2047 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 26894 \cdot ft \cdot k$$

Guess:  $L := \frac{2H_f - W_e}{2}$   $f_{max} := 4986 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \quad L_6 = 34.7 \text{ ft} \quad f_6 = 2706 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, L_6, 0 \cdot ft)$   $L_{b6} = 0.0 \cdot ft$

Maximum soil bearing pressure:  $f_{max6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, f_6, 0 \cdot psf)$   $f_{max6} = 0 \cdot psf$



#### 4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing ( $L_b$ ) equal to the full octagon width ( $H_f$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $f_{max}$  and the difference in maximum and minimum bearing pressures ( $df$ ).

$$F := W_W \quad F = 2047 \cdot k \quad M_{toe} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 26894 \cdot ft \cdot k$$

Guess:

$$f_{max} := 4611 \cdot psf$$

$$df := 6072 \cdot psf$$

Given

$$F = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\left( \frac{df}{f_{max}} \right) := \text{Find}(df, f_{max})$$

$$f_{max} = 3243 \cdot psf$$

$$df = 4509 \cdot psf$$

If the solution does not converge to the assumed pressure distribution, then the value of bearing length and soil pressure is set to zero.

$$\text{Maximum soil bearing pressure:} \quad f_{max7} := \text{if}(f_{max} - df < 0 \cdot psf, 0 \cdot psf, f_{max}) \quad f_{max7} = 0 \cdot psf$$

$$\text{Minimum soil bearing pressure:} \quad f_{min7} := \text{if}(f_{max7} > 0, f_{max7} - df, 0 \cdot psf) \quad f_{min7} = 0 \cdot psf$$

### D. Bearing Length Check - Extreme Loading

*Select Bearing Length and Pressure Distribution*

Bearing length:  $L_{bW} := \text{if}(f_{\max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6})$   $L_{bW} = 30.2 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\max W} := \text{if}\left(L_{bW} < \frac{H_f}{2}, f_{\max4}, \text{if}(L_{bW} < H_f - W_e, f_{\max5}, \text{if}(L_{bW} < H_f, f_{\max6}, f_{\max7}))\right)$

$f_{\max W} = 4171 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min W} := \text{if}(L_{bW} < H_f, 0 \cdot \text{psf}, f_{\min7})$

$f_{\min W} = 0 \cdot \text{psf}$

Determine controlling load case:  $L_b := L_{bW}$

$L_b = 30.2 \text{ ft}$

Maximum soil bearing pressure:  $f_{\max} := f_{\max W}$

$f_{\max} = 4171 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min} := f_{\min W}$

$f_{\min} = 0 \cdot \text{psf}$

Percent of base by length in  
compression under extreme loading:  $\frac{L_b}{H_f} = 0.56$

$\text{if}\left(\frac{L_b}{H_f} \geq 0.5, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}$
---

(Reference 10)

## E. Foundation Volume and Weight Calculations - Normal Loading

Total dead weight:  $W_{\text{totN}} := W_p + W_f + W_s + W_N - F_b$   $W_{\text{totN}} = 2052 \cdot \text{k}$

Overturing moment:  $M_{\text{oN}} := \sqrt{(M_N + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_N = 14093 \cdot \text{k} \cdot \text{ft}$

## F. Soil Pressure Calculations - Normal Loading

### 1) Assumed Soil Pressure Case 1

Assume triangular distribution with length of bearing ( $L_b$ ) greater than  $W_e$  but less than or equal to half the major octagon width ( $H_f/2$ ).

Set  $F=W_{\text{totN}}$  and  $M=M_{\text{toe}}$ , and solve for  $L_b$  and  $f_{\text{max}}$

$F := W_{\text{totN}}$   $F = 2052 \cdot \text{k}$   $M_{\text{toe}} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{\text{oN}}$   $M_{\text{toe}} = 41421 \cdot \text{ft} \cdot \text{k}$

Guess:  $L := \frac{2W_e + H_f}{4}$   $f_{\text{max}} := 3036 \cdot \text{psf}$

Given

$$F = \int_0^{W_e} \left[ f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^L f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy$$

$$M_{\text{toe}} = \int_0^{W_e} \left[ f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^L f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy$$

$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \text{Find}(L, f_{\text{max}})$   $L_4 = 47.7 \text{ ft}$   $f_4 = 2156 \cdot \text{psf}$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, L_4, 0 \cdot \text{ft} \right)$   $L_{b4} = 0.0 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\text{max}4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, f_4, 0 \cdot \text{psf} \right)$   $f_{\text{max}4} = 0 \cdot \text{psf}$

## 2) Assumed Soil Pressure Case 2

Assume triangular distribution with length of bearing ( $L_b$ ) greater than half the major octagon width ( $H_f/2$ ) but less than or equal to difference between the full octagon width ( $H_f$ ) and  $W_e$ .

Set  $F=W_{totN}$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_{totN} \quad F = 2052 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 41421 \cdot ft \cdot k$$

Guess:  $L := \frac{3H_f - 2W_e}{4}$   $f_{max} := 2964 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left(y - \frac{H_f}{2}\right)}{S_f} \right] \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_5 = 50.9 \text{ ft} \quad f_5 = 2111 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, L_5, 0 \cdot ft \right) \quad L_{b5} = 0.0 \cdot ft$

Maximum soil bearing pressure:  $f_{max5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, f_5, 0 \cdot psf \right) \quad f_{max5} = 0 \cdot psf$

### 3) Assumed Soil Pressure Case 3

Assume triangular distribution with length of bearing ( $L_b$ ) greater than the difference between the full octagon width ( $H_f$ ) and  $W_e$  but less than the full octagon width ( $H_f$ ).

Set  $F=W_{totN}$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_{totN} \quad F = 2052 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 41421 \cdot ft \cdot k$$

Guess:  $L := \frac{2H_f - W_e}{2}$   $f_{max} := 2892 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \quad L_6 = 51.1 \text{ ft} \quad f_6 = 2107 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, L_6, 0 \cdot ft)$   $L_{b6} = 51.1 \cdot ft$

Maximum soil bearing pressure:  $f_{max6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, f_6, 0 \cdot psf)$   $f_{max6} = 2107 \cdot psf$

#### 4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing ( $L_b$ ) equal to the full octagon width ( $H_f$ ).

Set  $F=W_{\text{totN}}$  and  $M=M_{\text{toe}}$ , and solve for  $f_{\text{max}}$  and the difference in maximum and minimum bearing pressures ( $df$ ).

$$F := W_{\text{totN}} \quad F = 2052 \cdot \text{k} \quad M_{\text{toe}} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{\text{oN}} \quad M_{\text{toe}} = 41421 \cdot \text{ft} \cdot \text{k}$$

Guess:

$$f_{\text{max}} := 2904 \cdot \text{psf}$$

$$df := 2657 \cdot \text{psf}$$

Given

$$F = \int_0^{W_e} \left[ \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^{H_f} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{\text{toe}} = \int_0^{W_e} \left[ \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^{H_f} \left( f_{\text{max}} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\left( \frac{df}{f_{\text{max}}} \right) := \text{Find}(df, f_{\text{max}}) \quad f_{\text{max}} = 2105 \cdot \text{psf} \quad df = 2229 \cdot \text{psf}$$

If the solution does not converge to the assumed pressure distribution, then the value of bearing length and soil pressure is set to zero.

$$\text{Maximum soil bearing pressure:} \quad f_{\text{max}7} := \text{if}(f_{\text{max}} - df < 0 \cdot \text{psf}, 0 \cdot \text{psf}, f_{\text{max}}) \quad f_{\text{max}7} = 0 \cdot \text{psf}$$

$$\text{Minimum soil bearing pressure:} \quad f_{\text{min}7} := \text{if}(f_{\text{max}7} > 0, f_{\text{max}7} - df, 0 \cdot \text{psf}) \quad f_{\text{min}7} = 0 \cdot \text{psf}$$

## **G. Bearing Length Check - Normal Loading**

*Select Bearing Length and Pressure Distribution*

Bearing length:  $L_{bN} := \text{if}(f_{\max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6})$   $L_{bN} = 51.1 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\max} := \text{if}\left(L_{bN} < \frac{H_f}{2}, f_{\max4}, \text{if}(L_{bN} < H_f - W_e, f_{\max5}, \text{if}(L_{bN} < H_f, f_{\max6}, f_{\max7}))\right)$   
 $f_{\max} = 2107 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min} := \text{if}(L_{bN} < H_f, 0 \cdot \text{psf}, f_{\min7})$   $f_{\min} = 0 \cdot \text{psf}$

Area of base in compression under normal loading:  $A_N := S_f^2 + \frac{3}{2} \cdot S_f \cdot W_e + (L_{bN} - S_f - W_e) \left[ S_f - \frac{S_f(L_{bN} - S_f - W_e)}{2 \cdot W_e} \right]$   
 $A_N = 2048 \cdot \text{ft}^2$

Percent of base by area in compression under normal loading:  $\frac{A_N}{A} = 98.9\%$

$\text{if}\left(\frac{A_N}{A} \geq 0.994, \text{"OK"}, \text{"No Good"}\right) = \text{"No Good"}$	(Reference 10)
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## V. Bearing Capacity Evaluation

(Reference 8)

### A. Design Soil Bearing Pressure - Normal Loading

Design overturning moment:	$M_{dN} := M_{oN}$	$M_{dN} = 14093 \cdot \text{ft} \cdot \text{k}$
Design vertical load:	$V_{dN} := W_{\text{totN}}$	$V_{dN} = 2052 \cdot \text{k}$
Design load eccentricity:	$e_{dN} := \frac{M_{dN}}{V_{dN}}$	$e_{dN} = 6.87 \text{ ft}$
Circular radius of octagon:	$R := \frac{D}{2}$	$R = 25.00 \text{ ft}$
Effective soil area in bearing:	$A_{\text{effN}} := 2 \cdot \left[ \left( R^2 \right) \cdot \text{acos} \left( \frac{e_{dN}}{R} \right) - e_{dN} \cdot \sqrt{R^2 - e_{dN}^2} \right]$	$A_{\text{effN}} = 1285 \text{ ft}^2$
Ellipse soil width in bearing:	$b_{eN} := 2 \cdot (R - e_{dN})$	$b_{eN} = 36.3 \text{ ft}$
Ellipse soil length in bearing:	$l_{eN} := 2 \cdot R \cdot \sqrt{1 - \left( 1 - \frac{b_{eN}}{2 \cdot R} \right)^2}$	$l_{eN} = 48.1 \text{ ft}$
Effective soil length in bearing:	$l_{\text{effN}} := \sqrt{A_{\text{effN}} \cdot \frac{l_{eN}}{b_{eN}}}$	$l_{\text{effN}} = 41.3 \text{ ft}$
Effective soil width in bearing:	$b_{\text{effN}} := \frac{l_{\text{effN}}}{l_{eN}} \cdot b_{eN}$	$b_{\text{effN}} = 31.1 \text{ ft}$
Design bearing pressure:	$f_{dN} := \frac{V_{dN}}{A_{\text{effN}}}$	$f_{dN} = 1596 \cdot \text{psf}$

### B. Bearing Capacity Check - Normal Loading

Allowable bearing pressure:	$f_{\text{all}_N} := 3300 \text{ psf}$	(Reference 2)
Ratio of design bearing pressure to allowable bearing pressure:	$\frac{f_{dN}}{f_{\text{all}_N}} = 0.48$	



### C. Design Soil Bearing Pressure - Extreme Normal Loading

Design overturning moment:  $M_{dW} := \sqrt{(M_e + M_{align} \cdot \cos(\Delta))^2 + (M_{align} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_e = 22744 \cdot \text{k} \cdot \text{ft}$

Design vertical load:  $V_{dW} := W_p + W_f + W_{te} + W_s - F_b \quad V_{dW} = 2051 \cdot \text{k}$

Design load eccentricity:  $e_{dW} := \frac{M_{dW}}{V_{dW}} \quad e_{dW} = 11.09 \text{ ft}$

Circular radius of octagon:  $R = 25.00 \text{ ft}$

Effective soil area in bearing:  $A_{effW} := 2 \cdot \left[ (R^2) \cdot \text{acos}\left(\frac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}^2} \right] \quad A_{effW} = 892 \text{ ft}^2$

Ellipse soil width in bearing:  $b_{eW} := 2 \cdot (R - e_{dW}) \quad b_{eW} = 27.8 \text{ ft}$

Ellipse soil length in bearing:  $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eW}}{2 \cdot R}\right)^2} \quad l_{eW} = 44.8 \text{ ft}$

Effective soil length in bearing:  $l_{effW} := \sqrt{A_{effW} \cdot \frac{l_{eW}}{b_{eW}}} \quad l_{effW} = 37.9 \text{ ft}$

Effective soil width in bearing:  $b_{effW} := \frac{l_{effW}}{l_{eW}} \cdot b_{eW} \quad b_{effW} = 23.5 \text{ ft}$

Design bearing pressure:  $f_{dW} := \frac{V_{dW}}{A_{effW}} \quad f_{dW} = 2299 \cdot \text{psf}$

### D. Bearing Capacity Check - Extreme Normal Loading

Allowable bearing pressure:  $f_{allW} := f_{all\_N} \cdot \left(\frac{4}{3}\right) = 4400 \cdot \text{psf} \quad (\text{Reference 2})$

Ratio of design bearing pressure to allowable bearing pressure:  $\frac{f_{dW}}{f_{allW}} = 0.52$

### E. Design Soil Bearing Pressure - Extreme Abnormal Loading

Design overturning moment:  $M_{dA} := \sqrt{(M_a + M_{align} \cdot \cos(\Delta))^2 + (M_{align} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_a = 28504 \cdot k \cdot ft$

Design vertical load:  $V_{dA} := W_p + W_f + W_{ta} + W_s - F_b$   $V_{dA} = 2047 \cdot k$

Design load eccentricity:  $e_{dA} := \frac{M_{dA}}{V_{dA}}$   $e_{dA} = 13.92 \text{ ft}$

Circular radius of octagon:  $R = 25.00 \text{ ft}$

Effective soil area in bearing:  $A_{effA} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dA}}{R}\right) - e_{dA} \cdot \sqrt{R^2 - e_{dA}^2} \right]$   $A_{effA} = 647 \text{ ft}^2$

Ellipse soil width in bearing:  $b_{eA} := 2 \cdot (R - e_{dA})$   $b_{eA} = 22.2 \text{ ft}$

Ellipse soil length in bearing:  $l_{eA} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eA}}{2 \cdot R}\right)^2}$   $l_{eA} = 41.5 \text{ ft}$

Effective soil length in bearing:  $l_{effA} := \sqrt{A_{effA} \cdot \frac{l_{eA}}{b_{eA}}}$   $l_{effA} = 34.8 \text{ ft}$

Effective soil width in bearing:  $b_{effA} := \frac{l_{effA}}{l_{eA}} \cdot b_{eA}$   $b_{effA} = 18.6 \text{ ft}$

Design bearing pressure:  $f_{dA} := \frac{V_{dA}}{A_{effA}}$   $f_{dA} = 3165 \cdot psf$

### F. Bearing Capacity Check - Extreme Abnormal Loading

Allowable bearing pressure:  $f_{allA} := f_{all\_N} \cdot \left(\frac{4}{3}\right) \cdot \frac{\alpha_c}{\alpha_a} = 5400 \cdot psf$  (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure:  $\frac{f_{dA}}{f_{allA}} = 0.59$

### G. Design Soil Bearing Pressure - Earthquake Loading

Design overturning moment:	$M_{dEQ} := M_{oEQ}$	$M_{dEQ} = 14440 \cdot \text{ft} \cdot \text{k}$
Design vertical load:	$V_{dEQ} := (1 + E_v) \cdot W_{EQ}$	$V_{dEQ} = 2052 \cdot \text{k}$
Design load eccentricity:	$e_{dEQ} := \frac{M_{dEQ}}{V_{dEQ}}$	$e_{dEQ} = 7.04 \text{ ft}$
Circular radius of octagon:	$R = 25.00 \text{ ft}$	
Effective soil area in bearing:	$A_{effEQ} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^2} \right]$	$A_{effEQ} = 1269 \text{ ft}^2$
Ellipse soil width in bearing:	$b_{eEQ} := 2 \cdot (R - e_{dEQ})$	$b_{eEQ} = 35.9 \text{ ft}$
Ellipse soil length in bearing:	$l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eEQ}}{2 \cdot R}\right)^2}$	$l_{eEQ} = 48.0 \text{ ft}$
Effective soil length in bearing:	$l_{effEQ} := \sqrt{A_{effEQ} \cdot \frac{l_{eEQ}}{b_{eEQ}}}$	$l_{effEQ} = 41.2 \text{ ft}$
Effective soil width in bearing:	$b_{effEQ} := \frac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$	$b_{effEQ} = 30.8 \text{ ft}$
Design bearing pressure:	$f_{dEQ} := \frac{V_{dEQ}}{A_{effEQ}}$	$f_{dEQ} = 1617 \cdot \text{psf}$

### H. Bearing Capacity Check - Earthquake Loading

Allowable bearing pressure:  $f_{allEQ} := f_{all\_N} \cdot \left(\frac{4}{3}\right) \cdot \frac{\alpha_e}{\alpha_a} = 5400 \cdot \text{psf}$  (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure:  $\frac{f_{dEQ}}{f_{allEQ}} = 0.30$

## VI. Foundation Stiffness Evaluation - Single Layer Native Soil Sites

Depth of embedment  
(half of foundation embedment):

$$h := \frac{1}{2} \cdot h_c + h_b$$

$h = 45.00 \cdot \text{in}$

Area of the footing:

$$A = 2071 \text{ ft}^2$$

Area of footing in compression:

$$A_N = 2048 \text{ ft}^2$$

Equivalent circular  
radius of footing:

$$R_{\text{stff}} := \sqrt{\frac{A_N}{\pi}}$$

$R_{\text{stff}} = 25.5 \cdot \text{ft}$

Subsoil density:

$$\rho := 125 \cdot \frac{\text{lb}}{\text{ft}^3}$$

(Reference 2)

$$\omega := \rho \cdot g$$

$\omega = 125 \cdot \text{pcf}$

Design shear wave velocity  
for interval from 8 to 50 feet:

$$V_s := 637 \cdot \frac{\text{ft}}{\text{sec}}$$

(Reference 2)

Poisson ratio:

$$\nu := 0.47$$

(Reference 2)

Initial shear modulus:

$$G_o := \rho \cdot V_s^2$$

$G_o = 1576 \cdot \text{ksf}$

$$E_o := 2 \cdot (1 + \nu) \cdot G_o$$

$E_o = 4635 \cdot \text{ksf}$

Initial elastic modulus:

Shear modulus ratio:

$$\gamma := 1 - (1.0) \cdot \left( \frac{f_{dN}}{3 \min(f_{allA}, f_{allW})} \right)^{0.3} = 0.47$$

(Reference 9)

Shear modulus:

$$G := \gamma \cdot G_o$$

$G = 740 \cdot \text{ksf}$

Elastic modulus:

$$E := 2 \cdot (1 + \nu) \cdot G$$

$E = 2176 \cdot \text{ksf}$

Rotational embedment  
Coefficient:

$$\eta_{\psi} := 1 + 1.2 \cdot (1 - \nu) \cdot \frac{h}{R_{stff}} + 0.2 \cdot (2 - \nu) \cdot \left( \frac{h}{R_{stff}} \right)^3 \quad \eta_{\psi} = 1.09 \quad (\text{Reference 5})$$

Embedment coefficient:

$$\eta_x := 1 + 0.55 \cdot (2 - \nu) \cdot \frac{h}{R_{stff}} \quad \eta_x = 1.12 \quad (\text{Reference 5})$$

Rotational stiffness of soil:

$$K_{\psi dyn} := \frac{8 \cdot G \cdot R_{stff}^3}{3 \cdot (1 - \nu)} \cdot \eta_{\psi} \quad K_{\psi dyn} = 92 \cdot \frac{GN \cdot m}{rad} \quad (\text{Reference 5})$$

$$K_{\psi req} = 3.00 \times 10^7 \cdot \frac{kN \cdot m}{rad} \quad (\text{Reference 3})$$

Design check:

$$\frac{K_{\psi dyn}}{K_{\psi req}} = 3.07$$

Translational stiffness  
of foundation:

$$K_{x dyn} := \frac{32 \cdot (1 - \nu) \cdot G \cdot R_{stff}}{7 - 8 \cdot \nu} \cdot \eta_x \quad K_{x dyn} = 1622 \cdot \frac{kN}{mm} \quad (\text{Reference 8})$$

Required dynamic translational  
stiffness:

$$K_{x req} = 1.00 \times 10^6 \cdot \frac{kN}{m} \quad (\text{Reference 3})$$

Design check:

$$\frac{K_{x dyn}}{K_{x req}} = 1.62$$

## VII. Anchor Bolt Design

### A. Strength Reduction and Load Factors

(Reference 1a and Reference 3)

Normal extreme load factor:  $\alpha_c = 1.35$

Abnormal extreme load factor:  $\alpha_a = 1.10$

Determine controlling extreme load case:  $\alpha_w := \text{if}(\alpha_c \cdot M_e > \alpha_a \cdot M_a, \alpha_c, \alpha_a)$   $\alpha_w = 1.10$

$M := \text{if}(\alpha_c \cdot M_e > \alpha_a \cdot M_a, M_e, M_a)$   $M = 27175 \cdot k \cdot ft$

$H := \text{if}(\alpha_c \cdot M_e > \alpha_a \cdot M_a, H_e, H_a)$   $H = 111 \cdot k$

$W_t := \text{if}(\alpha_c \cdot M_e > \alpha_a \cdot M_a, W_{te}, W_{ta})$   $W_t = 457 \cdot k$

Beneficial dead load factor:  $\alpha_{d1} := 0.9$

Bearing Factor:  $\phi_{br} \equiv 0.65$

Anchor tension load factor:  $\alpha_{pt} := 1.2$

Fastener Factor:  $\phi_f := 0.75$  (Reference 1c)

Non-Beneficial dead load factor:  $\alpha_{d2} := 1.2$

Shear Factor:  $\phi_v \equiv 0.75$

Earthquake load factor:  $\alpha_{EQ} := 1.0$

Flexure Factor:  $\phi_b \equiv 0.90$

Alignment load factor:  $\alpha_{d3} := 1.0$

Beneficial EQ dead load factor:  $\alpha_{d1EQ} := 0.9 - E_v$   $\alpha_{d1EQ} = 0.90$  (Reference 1)

Non-Beneficial EQ dead load factor:  $\alpha_{d2EQ} := 1.2 + E_v$   $\alpha_{d2EQ} = 1.20$  (Reference 1)

Sagging side load case:  $\alpha_w \text{ Wind} + \alpha_{d1} \text{ Dead}$  where Dead is dead load of soil, concrete, turbine, and tower

Hogging side load case:  $\alpha_{d2} \text{ Dead}$  where Dead is dead load of soil and concrete and uplift edge resistance of soil

## **B. Embedment Ring Dimensions**

Flange width:	$w_{\text{flange}} = 10.94 \cdot \text{in}$	$w_{\text{flange}} = 278 \cdot \text{mm}$
Embedment ring width:	$w := w_{\text{flange}}$	$w = 10.94 \cdot \text{in}$
Embedment ring thickness:	$t := 1.0 \cdot \text{in}$	

## **C. Anchor Bolt Dimensions and Data**

### ASTMA615 Grade 75

Nominal anchor bolt diameter:	$d_b := 1.25 \cdot \text{in}$
Bolt area through minimum diameter of threads:	$A_b := 1.27 \cdot \text{in}^2$
Washer diameter.:	$d_n := 3 \cdot \text{in}$
Outside diameter of PVC bolt sleeve:	$d_{\text{SDR}} := 1.660 \text{in}$
Yield strength:	$F_{yb} := 75 \cdot \text{ksi}$
Tensile strength:	$F_t := 100 \cdot \text{ksi}$

## **D. Material Properties**

Concrete strength of pedestal:	$f_{cp} := 5000 \cdot \text{psi}$
Embedment ring yield strength:	$F_y := 36000 \cdot \text{psi}$
Embedment ring tensile strength:	$F_u := 58000 \cdot \text{psi}$

## E. Anchor Bolt Design

Design loss:

$$\mu := 20\%$$

Maximum unfactored moment on bolts:

$$M_{\text{bolt}} := \sqrt{(\max(M_e, M_a) + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} = 27670 \cdot \text{k} \cdot \text{ft}$$

Maximum factored moment on bolts:

$$M_{\text{ubolt}} := \sqrt{(\alpha_w \cdot M + \alpha_{d3} \cdot M_{\text{align}} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{\text{align}} \cdot \sin(\Delta))^2} = 30387 \cdot \text{k} \cdot \text{ft}$$

Maximum factored seismic moment on bolts:

$$M_{\text{usbolt}} := \sqrt{(\alpha_{EQ} \cdot M_{OE} + \alpha_{d3} \cdot M_{\text{align}} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{\text{align}} \cdot \sin(\Delta))^2} = 13999 \cdot \text{k} \cdot \text{ft}$$

Minimum pre-tension for wind loading (if used):

$$T_{\text{preW}} := \left( \frac{4 \cdot M_{\text{bolt}}}{N \cdot D_{\text{avg}}} - \frac{W_t}{N} \right) \cdot (1 + \mu) \quad T_{\text{preW}} = 63.7 \cdot \text{k}$$

Minimum pre-tension for fatigue loading:

$$T_{\text{preFAT}} := \left( \frac{4 \cdot \max(M_{\text{maxnorth}})}{N \cdot D_{\text{avg}}} - \frac{W_{\text{mean}}}{N} \right) \cdot (1 + \mu) \quad T_{\text{preFAT}} = 39.6 \cdot \text{k}$$

Set to  $T_{\text{pre}}$ :

$$T_{\text{pre}} := 72 \cdot \text{k}$$

$$\% \text{yield} := \frac{T_{\text{pre}}}{F_{yb} \cdot A_b} \quad \% \text{yield} = 76\%$$

$$\% \text{ultimate} := \frac{T_{\text{pre}}}{F_t \cdot A_b} \quad \% \text{ultimate} = 57\%$$

Wind load maximum factored tension load in anchor:

$$T_{\text{uW}} := \frac{4 \cdot M_{\text{ubolt}}}{N \cdot D_{\text{avg}}} - \frac{\alpha_{d1} \cdot W_t}{N} \quad T_{\text{uW}} = 59 \cdot \text{k}$$

Seismic load maximum factored tension load in anchor:

$$T_{\text{uEQ}} := \frac{4 \cdot M_{\text{usbolt}}}{N \cdot D_{\text{avg}}} - \frac{\alpha_{d1EQ} \cdot W_{OE}}{N} \quad T_{\text{uEQ}} = 26 \cdot \text{k}$$

Fatigue load maximum tension load in anchor:

$$T_{\text{uFAT}} := \frac{4 \cdot \max(M_{\text{maxnorth}})}{N \cdot D_{\text{avg}}} - \frac{W_{\text{mean}}}{N} \quad T_{\text{uFAT}} = 33 \cdot \text{k}$$

Determine controlling load case:

$$T_u := \max(T_{\text{uW}}, T_{\text{uEQ}}, T_{\text{uFAT}}) \quad T_u = 59 \cdot \text{kip}$$

Design tension strength:

$$\phi T_n := \min(\phi_F \cdot F_t \cdot A_b, \phi_b \cdot F_{yb} \cdot A_b) \quad \phi T_n = 86 \cdot \text{k} \quad (\text{Reference 1c})$$

Design check:

$$\frac{T_u}{\phi T_n} = 0.69 \quad \text{Shear stress in bolt is negligible and, therefore, is not included.}$$



## VIII. Bottom Flange Bearing, Grout, and Embedment Plate Connection Design

### A. Material Properties

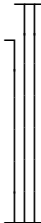
3-day grout strength:  $f_{c3} := 4500 \cdot \text{psi}$

28-day grout strength:  $f_{c28} := 8000 \cdot \text{psi}$

Grout thickness:  $t_{gr} := 0 \cdot \text{in}$

Bearing area of base flange:  $A_{br1} := \frac{\pi}{4} \cdot (\text{OD}^2 - \text{ID}^2) - N \cdot \frac{\pi \cdot d_{\text{SDR}}^2}{4}$   $A_{br1} = 5488 \cdot \text{in}^2$

Section modulus of base flange:  $S_1 := \frac{\pi}{32 \cdot \text{OD}} (\text{OD}^4 - \text{ID}^4) \dots$

$$+ \frac{2}{D_o} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{\text{SDR}}^4 + \frac{\pi}{2} \cdot d_{\text{SDR}}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \dots$$


$S_1 = 217587 \cdot \text{in}^3$

Area at bottom of grout:  $A_1 := w_{\text{flange}}$   $A_1 = 10.9 \cdot \text{in}$

Area of limiting bearing within grout:  $A_2 := w_{\text{flange}} + 2 \cdot t_{gr}$   $A_2 = 10.9 \cdot \text{in}$

$A := \min \left( \sqrt{\frac{A_2}{A_1}}, 2 \right)$   $A = 1.00$

### **B. Check 3 Day Grout Strength**

Design bearing strength:  $\phi b_{n3} := \phi_{br} \cdot 0.85 \cdot f_{c3} \cdot A$  (Reference 1a)  $\phi b_{n3} = 2.5 \cdot \text{ksi}$

Ultimate self weight bearing stress:  $b_{u3D} := \left( \alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{u3D} = 2.3 \cdot \text{ksi}$

Ultimate seismic bearing stress:  $b_{u3EQ} := \left( \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{u3EQ} = 2.3 \cdot \text{ksi}$

Determine controlling load case and check capacity:  $b_{u3} := \max(b_{u3D}, b_{u3EQ})$   $b_{u3} = 2.3 \cdot \text{ksi}$

$\frac{b_{u3}}{\phi b_{n3}} = 0.93$
-------------------------------------

### **C. Check 28 Day Grout Strength**

Design bearing strength:  $\phi b_{n28} := \phi_{br} \cdot 0.85 \cdot f_{c28} \cdot A$  (Reference 1a)  $\phi b_{n28} = 4.4 \cdot \text{ksi}$

Ultimate wind bearing stress:  $b_{u28W} := \alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \frac{M_{ubolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$   $b_{u28W} = 3.98 \cdot \text{ksi}$

Ultimate seismic bearing stress:  $b_{u28EQ} := \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{EQ} \cdot \frac{M_{usbolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$   $b_{u28EQ} = 3.1 \cdot \text{ksi}$

Determine controlling load case and check capacity:  $b_{u28} := \max(b_{u28W}, b_{u28EQ})$   $b_{u28} = 4.0 \cdot \text{ksi}$

$\frac{b_{u28}}{\phi b_{n28}} = 0.90$
---------------------------------------

### D. Check Bottom Flange Bearing on Concrete

Grout thickness:

$$t_g := 0 \cdot \text{in}$$

Pullout force due to wind:

$$P_{uW} := 2 \left( \frac{4M_{ubolt}}{N \cdot D_{avg}} - \alpha_{d1} \cdot \frac{W_t}{N} \right) \quad P_{uW} = 118 \cdot \text{k}$$

Pullout force due to seismic:

$$P_{uEQ} := 2 \left( \frac{4 \cdot M_{usbolt}}{N \cdot D_{avg}} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{N} \right) \quad P_{uEQ} = 51 \cdot \text{k}$$

Determine controlling load case:

$$P_u := \max(P_{uW}, P_{uEQ}) \quad P_u = 118 \cdot \text{k}$$

Bearing area at bottom of grout:

$$A_{br2} := \frac{\pi}{4} \cdot \left[ (OD + 2 \cdot t_g)^2 - (ID - 2 \cdot t_g)^2 \right] - N \cdot \frac{\pi \cdot d_{SDR}^2}{4} \quad A_{br2} = 5488 \cdot \text{in}^2$$

Section modulus at bottom of grout:

$$S_2 := \frac{\pi}{32 \cdot (OD + 2 \cdot t_g)} \left[ (OD + 2 \cdot t_g)^4 - (ID - 2 \cdot t_g)^4 \right] \dots$$

$$+ \left[ \frac{2}{(OD + 2 \cdot t_g)} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \dots \right. \right. \right. \right]$$

$$\left. \left. \left. + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \right]$$

$$S_2 = 217939 \cdot \text{in}^3$$

Ultimate wind stress:

$$b_{uW} := \frac{M_{ubolt}}{S_2} + \alpha_{d2} \cdot \frac{W_t}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}} \quad b_{uW} = 4.0 \cdot \text{ksi}$$

Ultimate seismic stress:

$$b_{uEQ} := \frac{M_{usbolt}}{S_2} + \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}} \quad b_{uEQ} = 3.1 \cdot \text{ksi}$$

Determine controlling load case:

$$b_u := \max(b_{uW}, b_{uEQ}) \quad b_u = 4.0 \cdot \text{ksi}$$

Check Bearing Plate Stresses on Concrete Due to Pre-tension and Extreme Wind Force:

Compute Areas being loaded: (Reference 1a)

Area at bottom of grout:

$$A_1 := w_{flange} + 2 \cdot t_g \quad A_1 = 10.9 \cdot \text{in}$$

Area of limiting bearing within concrete:

$$A_2 := A_1 + [C - (OD + 2 \cdot t_g)] \quad A_2 = 35.6 \cdot \text{in}$$

$$A := \min \left( \sqrt{\frac{A_2}{A_1}}, 2 \right) \quad A = 1.80$$

Design bearing strength:

$$\phi b_n := \phi_{br} \cdot 0.85 \cdot f_{cp} \cdot A \quad (\text{Reference 1a}) \quad \phi b_n = 5.0 \cdot \text{ksi}$$

$$\frac{b_u}{\phi b_n} = 0.80$$

## E. Check Pullout Strength of Embedment Ring/Anchor Bolt Connection

Conservatively ignored

## E2. Check Pullout Strength of Embedment Ring/Anchor Bolt Connection (concrete only)

Due to Wind:  $P_{uW} = 117.84 \cdot k$

Angle of subtended arc:  $\alpha_1 := \frac{4 \cdot \pi}{N}$   $\alpha_1 = 5.14 \cdot \text{deg}$

Depth of Outer Shear Cone:  $l_{eo} := \frac{h_b + h_c - h_e - (B - D_{avg} - w) \cdot \frac{1}{2}}{\frac{h_c}{a} + 1} \dots$   $l_{eo} = 49.2 \cdot \text{in}$   
 $+ (B - D_{avg} - w) \cdot \frac{1}{2}$

Area of Outer Shear Cone Segment:  $A_o := \frac{\alpha_1}{360 \cdot \text{deg}} \cdot \pi \cdot l_{eo} \cdot \sqrt{2} \cdot (OD + l_{eo})$   $A_o = 713.33 \cdot \text{in}^2$

Area of Bottom Plane:  $A_{bot} := \frac{\alpha_1}{360 \cdot \text{deg}} \cdot \pi \cdot \frac{ID^2}{4}$   $A_{bot} = 278.26 \cdot \text{in}^2$

Total Areas:  $A_{conc} := A_o + A_{bot}$   $A_{conc} = 991.59 \cdot \text{in}^2$

Pullout Strength of Outer Shear Cone Segment:  $\phi V_{n_p} := \phi_v \cdot A_o \cdot 2.8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \dots$   $\phi V_{n_p} = 164.95 \cdot \text{kip}$   
 $+ \phi_v \cdot A_{bot} \cdot 4 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c}$

$\frac{P_{uW}}{\phi V_{n_p}} = 0.71$
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## F. Check Bending Strength of Embedment Plate

Nut to nut circumferential distance:  $d_1 := \frac{2 \cdot D_o \cdot \pi}{N} - d_n$   $d_1 = 4.82 \cdot \text{in}$

Nut to nut radial distance:  $d_2 := \left( \frac{D_o - D_i}{2} \right) - d_n$   $d_2 = 2.83 \cdot \text{in}$

Edge distance:  $d_3 := \left( \frac{OD - D_o}{2} \right) - \frac{d_n}{2}$   $d_3 = 1.06 \cdot \text{in}$

Ultimate wind stress:  $b_{\text{uplate}W} := \frac{M_{\text{ubolt}}}{S_1} - \alpha_{d1} \cdot \frac{W_t}{A_{br1}}$   $b_{\text{uplate}W} = 1.6 \cdot \text{ksi}$

Ultimate seismic stress:  $b_{\text{uplate}EQ} := \frac{M_{\text{usbolt}}}{S_1} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{A_{br1}}$   $b_{\text{uplate}EQ} = 0.7 \cdot \text{ksi}$

Determine controlling load case:  $b_{\text{uplate}} := \max \left( b_{\text{uplate}W}, b_{\text{uplate}EQ}, \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{\text{uplate}} = 2.2 \cdot \text{ksi}$

Plastic section modulus per inch:  $Z_y := \frac{t^2}{4}$   $Z_y = 0.25 \cdot \frac{\text{in}^3}{\text{in}}$

Section modulus per inch:  $S_y := \frac{t^2}{6}$   $S_y = 0.17 \cdot \frac{\text{in}^3}{\text{in}}$

*Check circumferential nut to nut bending*

$M_{u.e1} := b_{\text{uplate}} \cdot \frac{d_1^2}{12} = 4.27 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$   $\phi M_n := \phi_b \cdot \min(F_y \cdot Z_y, 1.6 \cdot F_y \cdot S_y) = 8.10 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e1}}{\phi M_n} = 0.53$

*Check circumferential nut to nut bending at splice*

$M_{u.e2} := b_{\text{uplate}} \cdot \frac{d_1^2}{8} = 6.40 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e2}}{\phi M_n} = 0.79$

*Check radial nut to nut bending*

$M_{u.e3} := b_{\text{uplate}} \cdot \frac{d_2^2}{12} = 1.47 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e3}}{\phi M_n} = 0.18$

*Check nut to edge bending*

$M_{u.e4} := \frac{b_{\text{uplate}}}{2} \cdot d_3^2 = 1.24 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e4}}{\phi M_n} = 0.15$

*Check shear rupture of washer through plate*

$V_{uj} := \max \left( \alpha_{pt} \cdot T_{pre}, \frac{P_u}{2} \right) = 86.40 \cdot \text{k}$   $\phi V_n := \phi_v \cdot \pi \cdot d_n \cdot t \cdot 0.6 \cdot F_u = 245.99 \cdot \text{k}$  (Reference 1c)  $\frac{V_{uj}}{\phi V_n} = 0.35$

## IX-a. Concrete Design - Extreme Loads

### A. Design Functions

Function describing the volume of concrete for each slice of the moment/shear calculations.

$$\text{ConcreteVolume}(y) := \begin{cases} h_b \cdot (B + 2 \cdot y) + \frac{y}{a} \cdot h_c \cdot (B + y) & \text{if } y \leq a \\ h_b \cdot (D) + h_c \cdot (B + a) & \text{otherwise} \end{cases}$$

Functions describing the weight of the soil wedge pieces acting on each slice of the moment/shear calculations.

$$\text{StaticSoilWedgeWeight}(\gamma_{sd}, \gamma_{ss}) := \begin{cases} \gamma_{sd} \cdot \frac{B \cdot \tan(\theta)}{2} \cdot (h_s - h_b)^2 & \text{if } d_{GWT} \geq h_s - h_b \\ \frac{B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{otherwise} \end{cases}$$

$$\text{VariableSoilWedgeWeight}(y, \gamma_{sd}, \gamma_{ss}) := \begin{cases} 0 & \text{if } d_{GWT} \geq h_s - h_b \\ \text{otherwise} \\ \begin{cases} \sqrt{2} \cdot \tan(\theta) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{if } y \leq a \\ \tan(\theta) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{otherwise} \end{cases} \end{cases}$$

Function describing the volume of dry soil over each slice of the moment/shear calculations.

$$\text{DrySoilVolume}(h_j, y) := \begin{cases} \text{if } d_{\text{GWT}} \geq h_s - h_b \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \frac{y^2 \cdot h_c}{a} + \sqrt{2} \cdot \tan(\theta) \cdot (h_s - h_b)^2 \right] & \text{if } y \leq a \\ D \cdot \left[ (h_s - h_b) - h_c \right] + h_c \cdot a + \tan(\theta) \cdot (h_s - h_b)^2 & \text{otherwise} \\ \text{if } d_{\text{GWT}} \leq h_s - h_j \\ d_{\text{GWT}} \cdot (B + 2 \cdot y) & \text{if } y \leq a \\ D \cdot d_{\text{GWT}} & \text{otherwise} \\ \text{otherwise} \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \left[ \frac{y^2 \cdot h_c}{a} - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 \right] \right] & \text{if } y \leq a \\ \left[ D \cdot (h_s - h_b - h_c) + \left[ h_c \cdot a - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 \right] \right] & \text{otherwise} \end{cases}$$

Function describing the volume of saturated soil over each slice of the moment/ shear calculations.

$$\text{SaturatedSoilVolume}(h_j, y) := \begin{cases} 0 & \text{if } d_{\text{GWT}} \geq h_s - h_b \\ \text{if } d_{\text{GWT}} \leq h_s - h_j \\ \left( (B + 2 \cdot y) \cdot \left( h_s - h_b - \frac{y}{a} \cdot h_c - d_{\text{GWT}} \right) + \frac{y^2 \cdot h_c}{a} \right) & \text{if } y \leq a \\ (h_s - h_b - h_c - d_{\text{GWT}}) \cdot D + h_c \cdot a & \text{otherwise} \\ \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 & \text{otherwise} \end{cases}$$

Function describing the effect of groundwater on the material weights over each slice of the moment/ shear calculations.

$$\text{BuoyancyWeight}(y) := \begin{cases} 0 & \text{if } d_{\text{GWT}} \geq h_s \\ \text{if } d_{\text{GWT}} < h_s \\ (B + 2 \cdot y) \cdot (h_s - d_{\text{GWT}}) & \text{if } y \leq a \\ (h_s - d_{\text{GWT}}) \cdot D & \text{otherwise} \end{cases}$$

## **B. Design Soil Bearing Pressure Wind Loading**

(Reference 8)

Design overturning moment:  $M_{dW} := \sqrt{(\alpha_w \cdot M + \alpha_{d3} \cdot M_{align} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{align} \cdot \sin(\Delta))^2} + \alpha_w \cdot (h_b + h_c + h_p) \cdot H$

$$M_{dW} = 31305 \cdot \text{k} \cdot \text{ft}$$

Design vertical load:  $V_{dW} := \alpha_{d1} \cdot \left( W_p + W_f + W_s - \frac{F_b}{\alpha_{d1}} + W_t \right)$

$$V_{dW} = 1792 \cdot \text{k}$$

Design load eccentricity:  $e_{dW} := \frac{M_{dW}}{V_{dW}}$

$$e_{dW} = 17.46 \text{ ft}$$

Circular radius of octagon:  $R := \frac{D}{2}$

$$R = 25.00 \text{ ft}$$

Effective soil area in bearing:  $A_{effW} := 2 \cdot \left[ (R^2) \cdot \text{acos}\left(\frac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}^2} \right]$

$$A_{effW} = 372 \cdot \text{ft}^2$$

Ellipse soil width in bearing:  $b_{eW} := 2 \cdot (R - e_{dW})$

$$b_{eW} = 15.1 \text{ ft}$$

Ellipse soil length in bearing:  $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eW}}{2 \cdot R}\right)^2}$

$$l_{eW} = 35.8 \text{ ft}$$

Effective soil length in bearing:  $l_{effW} := \sqrt{A_{effW} \cdot \frac{l_{eW}}{b_{eW}}}$

$$l_{effW} = 29.7 \text{ ft}$$

Design bearing pressure:  $f_{dW} := \frac{V_{dW}}{A_{effW}}$

$$f_{dW} = 4820 \cdot \text{psf}$$

Effective soil width in bearing:  $b_{effW} := \frac{l_{effW}}{l_{eW}} \cdot b_{eW}$

$$b_{effW} = 12.5 \text{ ft}$$

$$x_{startW} := \frac{D}{2} - e_{dW} - \frac{b_{effW}}{2}$$

$$x_{startW} = 1.28 \text{ ft}$$



### C. Design Soil Bearing Pressure Seismic Loading

(Reference 8)

Design overturning moment: 
$$M_{dEQ} := \sqrt{(\alpha_{EQ} \cdot M_{OE} + \alpha_{d3} \cdot M_{align} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{align} \cdot \sin(\Delta))^2} + \alpha_{EQ} \cdot (h_b + h_c + h_p) \cdot H_{OE}$$

$M_{dEQ} = 14440 \cdot \text{ft} \cdot \text{k}$

Design vertical load: 
$$V_{dEQ} := \alpha_{d1EQ} \cdot \left( W_p + W_f + W_s - \frac{F_b}{\alpha_{d1EQ}} + W_{OE} \right)$$

$V_{dEQ} = 1796 \cdot \text{kip}$

Design load eccentricity: 
$$e_{dEQ} := \frac{M_{dEQ}}{V_{dEQ}}$$

$e_{dEQ} = 8.04 \text{ ft}$

Effective soil area in bearing: 
$$A_{effEQ} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^2} \right]$$

$A_{effEQ} = 1174 \cdot \text{ft}^2$

Ellipse soil width in bearing: 
$$b_{eEQ} := 2 \cdot (R - e_{dEQ})$$

$b_{eEQ} = 33.9 \text{ ft}$

Ellipse soil length in bearing: 
$$l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eEQ}}{2 \cdot R}\right)^2}$$

$l_{eEQ} = 47.3 \text{ ft}$

Effective soil length in bearing: 
$$l_{effEQ} := \sqrt{A_{effEQ} \cdot \frac{l_{eEQ}}{b_{eEQ}}}$$

$l_{effEQ} = 40.5 \text{ ft}$

Design bearing pressure: 
$$f_{dEQ} := \frac{V_{dEQ}}{A_{effEQ}}$$

$f_{dEQ} = 1530 \cdot \text{psf}$

Effective soil width in bearing: 
$$b_{effEQ} := \frac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$$

$b_{effEQ} = 29.0 \text{ ft}$

$$x_{startEQ} := \frac{D}{2} - e_{dEQ} - \frac{b_{effEQ}}{2}$$

$x_{startEQ} = 2.46 \text{ ft}$

## D. Structural Calculations

Area of pedestal:  $A_{ped} := \pi \cdot \frac{C^2}{4}$   $A_{ped} = 227 \cdot \text{ft}^2$

Equivalent square dimension:  $S_{ped} := \sqrt{A_{ped}}$   $S_{ped} = 15.1 \text{ ft}$

Distance to critical section:  $x_{face} := \frac{D - S_{ped}}{2}$   $x_{face} = 17.5 \text{ ft}$

Number of section slices to be taken within "a":  $n := \text{trunc}\left(\frac{a}{\text{ft}} \cdot 2\right)$   $n = 29$

Number of section slices to be taken beyond "a":  $n_j := \text{trunc}\left(\frac{x_{face}}{\text{ft}} \cdot 2\right) - n$   $n_j = 5.00$

Sloped portion of footing:  $i := 1, 2 \dots n$

From a to the critical section ( $x_{face}$ ):  $j := n + 1, n + 2 \dots n + n_j$

Array counter for all slices:  $q := 1, 2 \dots n + n_j$

Plan location of section:  $x_i := \frac{i}{2} \cdot \text{ft}$

$$x_j := \max(x) + (x_{face} - \max(x)) \cdot \frac{j - n}{n_j}$$

Height of section:  $h_i := h_b + \frac{x_i}{a} \cdot h_c$   $h_j := h_b + h_c$

Depth, d, as a function of distance along the sloped portion of the foundation is:  $d_i := h_b + \frac{x_i}{a} \cdot h_c - 3.75 \cdot \text{in}$

$$d_j := h_b + h_c - 3.75 \cdot \text{in}$$

Equivalent square dimension of embedment ring:  $S_{embedring} := \sqrt{\pi \cdot \frac{OD^2}{4}}$   $S_{embedring} = 13.2 \text{ ft}$

Alternate distance to critical section based on edge of embedment ring:  $x_{face\_alt} := \frac{D - S_{embedring}}{2}$   $x_{face\_alt} = 18.4 \text{ ft}$

Plan location (alternate) of section:  $x_{alt_i} := \frac{i}{2} \cdot \text{ft}$

$$x_{alt_j} := \max(x_{alt}) + (x_{face\_alt} - \max(x_{alt})) \cdot \frac{j - n}{n_j}$$

The exact solution for the factored shear force under wind loading due to soil bearing pressure along the sloped portion of the foundation is:

Design Shear from edge of footing to just before  $b_{effW}$

$$V_{uW_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startW} \\ \int_{x_{startW}}^{\min(x_i, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \, dy & \text{otherwise} \end{cases} + \int_0^{x_i} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_i, y) \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \dots \end{aligned} \right) dy \dots \right. \\ \left. + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \right)$$

Design Shear from a to  $x_{face}$

$$V_{uW_j} := \int_{x_{startW}}^{\min(x_j, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \, dy + \int_0^a \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \dots \end{aligned} \right) dy \dots \right. \\ \left. + \int_a^{x_j} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \dots \end{aligned} \right) dy + \left( -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \dots \right. \right. \\ \left. \left. + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \right) \right)$$

The exact solution for the factored shear force under seismic loading due to soil bearing pressure along the sloped portion of the foundation is:

Design Shear from edge of footing to just before  $b_{effEQ}$

$$V_{uEQ_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startEQ} \\ \int_{x_{startEQ}}^{\min(x_i, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ} \, dy & \text{otherwise} \end{cases} + \int_0^{x_i} -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy \dots$$

$$+ -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w)$$

Design Shear from a to  $x_{face}$

$$V_{uEQ_j} := \int_{x_{startEQ}}^{\min(x_j, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ} \, dy + \int_0^a -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy \dots$$

$$+ \int_a^{x_j} -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \dots$$

$$+ \text{StaticSoilWedgeWeight}(0pcf, \gamma_w)$$



*Bottom Reinforcing Design Moments*

Solution for the design bending moment due to soil bearing pressure under wind loading is:

$$\begin{aligned}
 M_{ubot1W_i} := & \begin{cases} 0 & \text{if } x_i \leq x_{startW} \\ \int_{x_{startW}}^{\min(x_i, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_i - y) \, dy & \text{otherwise} \end{cases} + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_i \dots \\
 & + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_i \dots \\
 & + \int_{x_i}^{x_i} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_i - y) \, dy \right) \\
 M_{ubot1W_j} := & \int_{x_{startW}}^{\min(x_j, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_j - y) \, dy + \int_0^a \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_j - y) \, dy \dots \right) \\
 & + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_j + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_j \dots \\
 & + \int_a^{x_j} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_j - y) \, dy \right)
 \end{aligned}$$

Solution for the design bending moment due to soil bearing pressure under seismic loading is:

$$M_{ubot1EQ_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startEQ} \dots \\ \int_{x_{startEQ}}^{\min(x_i, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ}(x_i - y) dy & \text{otherwise} \\ + \int_0^{x_i} \left( -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_i - y) dy \dots \\ + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_i \dots \end{cases}$$

$$M_{ubot1EQ_j} := \begin{cases} \int_{x_{startEQ}}^{\min(x_j, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ}(x_j - y) dy \dots \\ + \int_0^a \left( -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_j - y) dy \dots \\ + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_j \dots \\ + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_j \dots \\ + - \int_0^{x_j} \left( \alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_j - y) dy \end{cases}$$

Determine controlling load case for bottom moments:  $^c M_{ubot1}_q := \max(M_{ubot1W}_q, M_{ubot1EQ}_q)$

*Top Reinforcing Design Moments*

The solution for the design bending moment due to the weight of concrete and soil above the footing and soil resistance along edge of footing is:

$$M_{utop1_i} := \int_0^{x_{alt_i}} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_i} - y) dy \dots$$

$$+ \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \text{StaticSoilWedgeWeight}(\gamma_{sstop}, \gamma_{sstop}) \cdot x_{alt_i}$$

$$M_{utop1_j} := \int_0^a \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_j} - y) dy \dots$$

$$+ \int_a^{x_{alt_j}} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_j} - y) dy \dots$$



### **E. Top and Bottom Reinforcing Selection**

Top middle bars:                       $\text{Size}_{\text{tmb}} := 6$                        $s_{\text{topm}} := 8 \cdot \text{in}$

Top outside bars:                       $\text{Size}_{\text{tob}} := 6$                        $s_{\text{topo}} := 8 \cdot \text{in}$

Bottom middle bars:                       $\text{Size}_{\text{bmb}} := 11$                        $s_{\text{botm}} := 9 \cdot \text{in}$

Bottom outside bars:                       $\text{Size}_{\text{bob}} := 7$                        $s_{\text{boto}} := 9 \text{in}$

Distance from centerline that defines  
middle/outside boundary:                       $W_m := \frac{C}{2} + 1.5 \cdot (h_b + h_c)$                        $W_m = 16.25 \text{ ft}$

### **F. Bar Cutoff Locations**

Distance of top bar cutoff from  
edge of footing:                       $cd_{\text{top}} := 5 \cdot \text{ft}$

Distance of bottom bar cutoff from  
edge of footing:                       $cd_{\text{bot}} := 0 \cdot \text{ft}$





Assign properties using lookup function, depending on bar size.

$$d_{i_{topm}} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{topm}} = 0.750 \cdot \text{in}$$

$$d_{i_{topo}} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{topo}} = 0.750 \cdot \text{in}$$

$$d_{i_{botm}} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{botm}} = 1.410 \cdot \text{in}$$

$$d_{i_{boto}} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{boto}} = 0.875 \cdot \text{in}$$

$$A_{topm} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{topm} = 0.44 \cdot \text{in}^2$$

$$A_{topo} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{topo} = 0.44 \cdot \text{in}^2$$

$$A_{botm} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{botm} = 1.56 \cdot \text{in}^2$$

$$A_{boto} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{boto} = 0.60 \cdot \text{in}^2$$

$$W_{topm} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{topm} = 1.502 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{topo} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{topo} = 1.502 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{botm} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{botm} = 5.313 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{boto} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{boto} = 2.044 \cdot \frac{\text{lbf}}{\text{ft}}$$

## **G. Bottom Reinforcing Development Length Past Critical Section**

### 1) Middle Bars

Rebar yield strength:  $f_{yB} = 75000 \cdot \text{psi}$

28 day concrete strength:  $f_c = 5000 \cdot \text{psi}$

Bottom middle layer bar spacing:  $s_{\text{botm}} = 9.00 \cdot \text{in}$

Bottom middle layer bar diameter:  $d_{i\text{botm}} = 1.410 \cdot \text{in}$

Reinforcement location factor:  $\alpha := 1.0$  <12 inches of concrete cast below (Reference 1a)

Coating factor:  $\beta := 1.0$  uncoated

Reinforcement size factor:  $\gamma := \text{if}(d_{i\text{botm}} < 0.875 \cdot \text{in}, 0.8, 1.0)$   $\gamma = 1.0$

Lightweight concrete factor:  $\lambda := 1.0$  normal weight concrete

Spacing factor:  $c := \min\left(\frac{\min\left(\frac{s_{\text{botm}}}{2}, c_{\text{bot}} + \frac{d_{i\text{botm}}}{2}\right)}{d_{i\text{botm}}}, 2.5\right)$   $c = 2.5$

Tension development length past critical section:  $l_{\text{dbotm}} := \frac{3 \cdot f_{yB} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c}} \cdot d_{i\text{botm}}$   $l_{\text{dbotm}} = 45 \cdot \text{in}$  (Reference 1a)

### 2) Outside Bars

Bottom outside layer bar spacing:  $s_{\text{boto}} = 9.00 \cdot \text{in}$

Bottom outside layer bar diameter:  $d_{i\text{boto}} = 0.875 \cdot \text{in}$

Reinforcement size factor:  $\gamma := \text{if}(d_{i\text{boto}} < 0.875 \cdot \text{in}, 0.8, 1.0)$   $\gamma = 1.0$

Spacing factor:  $c := \min\left(\frac{\min\left(\frac{s_{\text{boto}}}{2}, c_{\text{bot}} + \frac{d_{i\text{boto}}}{2}\right)}{d_{i\text{boto}}}, 2.5\right)$   $c = 2.5$

Tension development length past critical section:  $l_{\text{dboto}} := \frac{3 \cdot f_{yB} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c}} \cdot d_{i\text{boto}}$   $l_{\text{dboto}} = 28 \cdot \text{in}$  (Reference 1a)

## H. Top Reinforcing Development Length Past Critical Section

### 1) Middle Bars

Top middle layer bar spacing:	$s_{topm} = 8.00 \cdot \text{in}$		
Top middle layer bar size:	$d_{i_{topm}} = 0.750 \cdot \text{in}$		
Reinforcement location factor:	$\alpha := 1.3$	>12 inches of concrete cast below	(Reference 1a)
Coating factor:	$\beta := 1.0$	uncoated	
Reinforcement size factor:	$\gamma := \text{if}(d_{i_{topm}} < 0.875 \cdot \text{in}, 0.8, 1.0)$	$\gamma = 0.8$	
Lightweight concrete factor:	$\lambda := 1.0$	normal weight concrete	
Spacing factor:	$c := \min\left(\frac{\min\left(\frac{s_{topm}}{2}, cc_{top} + \frac{d_{i_{topm}}}{2}\right)}{d_{i_{topm}}}, 2.5\right)$	$c = 2.5$	
Tension development length past critical section:	$l_{d_{topm}} := \frac{3 \cdot f_{yT} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c \cdot \text{psi}}} \cdot d_{i_{topm}}$	$l_{d_{topm}} = 25 \cdot \text{in}$	(Reference 1a)

### 2) Outside Bars

Top outside layer bar spacing:	$s_{topo} = 8.00 \cdot \text{in}$		
Top outside layer bar size:	$d_{i_{topo}} = 0.750 \cdot \text{in}$		
Reinforcement size factor:	$\gamma := \text{if}(d_{i_{topo}} < 0.875 \cdot \text{in}, 0.8, 1.0)$	$\gamma = 0.8$	
Spacing factor:	$c := \min\left(\frac{\min\left(\frac{s_{topo}}{2}, cc_{top} + \frac{d_{i_{topo}}}{2}\right)}{d_{i_{topo}}}, 2.5\right)$	$c = 2.5$	
Tension development length past critical section:	$l_{d_{topo}} := \frac{3 \cdot f_{yT} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c \cdot \text{psi}}} \cdot d_{i_{topo}}$	$l_{d_{topo}} = 25 \cdot \text{in}$	

## I. Calculate Actual Bottom Moment Capacity

Width of footing at section:

$$W_{bot\ q} := \text{if}(q \leq n, B + 2 \cdot x_{q}, D)$$

Number of bars within middle section:

$$n_{botm\ q} := \text{if}\left(W_m < \frac{W_{bot\ q}}{2}, \text{trunc}\left(\frac{W_m}{s_{botm}}\right), \text{trunc}\left(\frac{\frac{W_{bot\ q}}{2} - \sqrt{2} \cdot cc_{top}}{s_{botm}}\right)\right)$$

Spacing of first bar beyond the middle/outside boundary line:

$$s_{b1bar\ q} := \text{if}\left(W_m < \frac{W_{bot\ q}}{2}, s_{botm} \cdot n_{botm\ q} + s_{boto} - W_m, 0.0\text{in}\right)$$

Number of bars across bottom of footing at section:

$$n_{bot\ q} := \begin{cases} n_{botm\ q} + \left\lceil \text{trunc}\left[\frac{0.5(W_{bot\ q} - 2W_m) - s_{b1bar\ q} - cc_{top}}{s_{boto}}\right] + 1 \right\rceil & \text{if } W_m < \frac{W_{bot\ q}}{2} \\ n_{botm\ q} & \text{otherwise} \end{cases}$$

Bar counter:

$$ib := 1, 2 \dots n_{bot\ n+n_j}$$

Distance of bars from centerline across bottom of footing:

$$z_{bot,ib} := \text{if}\left[ib \leq n_{botm\ n+n_j}, ib \cdot s_{botm}, s_{botm} \cdot n_{botm\ n+n_j} + s_{boto} \cdot (ib - n_{botm\ n+n_j})\right]$$

Depth of footing for bottom middle steel at point:

$$d_{botm_i} := h_b + \frac{x_i}{a} \cdot h_c - cc_{bot} - di_{botm}$$

$$d_{botm_j} := h_b + h_c - cc_{bot} - di_{botm}$$

Depth of footing for bottom outside steel at point:

$$d_{boto_i} := h_b + \frac{x_i}{a} \cdot h_c - cc_{bot} - di_{boto}$$

$$d_{boto_j} := h_b + h_c - cc_{bot} - di_{boto}$$

Depth of each bar at section:

$$d_{barb_{q,ib}} := \begin{cases} \text{if } ib \cdot s_{botm} > \frac{B}{2} \\ \left| \begin{array}{l} d_{botm\ q} - \left(z_{bot,ib} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{if } ib \leq n_{botm\ q} \\ d_{boto\ q} - \left(z_{bot,ib} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{otherwise} \end{array} \right. \\ d_{botm\ q} \quad \text{otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{sbotm\ q} := \text{if}\left(x_q \geq cd_{bot}, n_{botm\ q} \cdot 2A_{botm}, \frac{1}{2} \cdot n_{botm\ q} \cdot 2A_{botm}\right) + A_{botm}$$

Area of steel provided across section at outside section:

$$A_{sboto\ q} := \text{if}\left[x_q \geq cd_{bot}, (n_{bot\ q} - n_{botm\ q}) \cdot 2 \cdot A_{boto}, \frac{1}{2} \cdot (n_{bot\ q} - n_{botm\ q}) \cdot 2 \cdot A_{boto}\right]$$

Applying ACI minimum reinforcing requirements:

$$\rho_{B,min} := \max\left(0.0014, \frac{0.0018 \cdot 60000 \cdot \text{psi}}{f_{yB}}\right) \quad \rho_{B,min} = 0.00144$$

Minimum area of steel  
required at section:

$$A_{sminb_q} := \rho_{B,min} \cdot \text{ConcreteVolume}(x_q) \quad (\text{Reference 1a})$$

Factored moment considering  
minimum reinforcing  
requirements at section:

$$M_{ubot_q} := \text{if}\left(A_{sbotm_q} + A_{sboto_q} \leq A_{sminb_q}, \text{"NG - BOTTOM R/F RATIO"}, \max(M_{ubot1_q}, 0)\right)$$

Footing is separated into strips  
containing one bar each. depth of  
compression block for each strip:

$$a_{botm} := \frac{A_{botm} \cdot f_{yB}}{0.85 \cdot f_c \cdot s_{botm}}$$

Depth of compression block for  
each outside strip:

$$a_{boto} := \frac{A_{boto} \cdot f_{yB}}{0.85 \cdot f_c \cdot s_{boto}}$$

Distance from section to end of bar  
for continuous bars:

$$l_{bot1_{q,ib}} := \text{if}\left[z_{bot_{ib}} > \frac{B}{2}, \max\left[x_q - \left(z_{bot_{ib}} - \frac{B}{2}\right) - \sqrt{2} \cdot cc_{top}, 0\right], \max(x_q - cc_{top}, 0)\right]$$

Distance from section to end of bar  
for cutoff bars:

$$l_{bot2_{q,ib}} := \max(x_q - cd_{bot}, 0)$$

Selection of appropriate bar end  
distance for bar in question:

$$l_{bot_{q,ib}} := \text{if}\left(ib \cdot s_{botm} - \frac{B}{2} < cd_{bot} \wedge \frac{ib}{2} \neq \text{trunc}\left(\frac{ib}{2}\right), l_{bot2_{q,ib}}, l_{bot1_{q,ib}}\right)$$

Factored moment capacity at section:

$$\phi M_{nbot_q} := \phi_b \cdot \left[ A_{botm} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,l}}}{l_{dbotm}}, 1\right) \cdot \left(d_{botm_q} - \frac{a_{botm}}{2}\right) + 2 \cdot \sum_{kk=1}^{n_{botm_q}} \left[ A_{botm} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,kk}}}{l_{dbotm}}, 1\right) \cdot \left(d_{barb_{q,kk}} - \frac{a_{botm}}{2}\right) \right] \dots \right. \\ \left. + 2 \cdot \sum_{jj=(n_{botm_q})+1}^{n_{bot_q}} \left[ A_{boto} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,jj}}}{l_{dboto}}, 1\right) \cdot \left(d_{barb_{q,jj}} - \frac{a_{boto}}{2}\right) \right] \right]$$

Check of factored moment vs.  
moment capacity at each section:

$$\text{check}_{bot_q} := \frac{M_{ubot_q}}{\phi M_{nbot_q}}$$

## J. Calculate Actual Top Moment Capacity

Applying ACI minimum reinforcing requirements:

$$\rho_{T,\min} := \max\left(\frac{3 \cdot \text{psi}^{0.5} \cdot \sqrt{f_c}}{f_{yT}}, \frac{200 \cdot \text{psi}}{f_{yT}}\right) \quad \rho_{T,\min} = 0.0028$$

Width of footing at section:

$$W_{\text{top}_q} := \text{if}(q \leq n, B + 2 \cdot x_{\text{alt}_q}, D)$$

Number of bars within middle section:

$$n_{\text{topm}_q} := \text{if}\left(W_m < \frac{W_{\text{top}_q}}{2}, \text{trunc}\left(\frac{W_m}{s_{\text{topm}}}\right), \text{trunc}\left(\frac{\frac{W_{\text{top}_q}}{2} - \sqrt{2} \cdot \text{cc}_{\text{top}}}{s_{\text{topm}}}\right)\right)$$

Spacing of first bar beyond the middle/outside boundary line:

$$s_{\text{t1bar}_q} := \text{if}\left(W_m < \frac{W_{\text{top}_q}}{2}, s_{\text{topm}} \cdot n_{\text{topm}_q} + s_{\text{topo}} - W_m, 0.0\text{in}\right)$$

Number of bars across top of footing at section:

$$n_{\text{top}_q} := \begin{cases} n_{\text{topm}_q} + \left\lceil \frac{0.5(W_{\text{top}_q} - 2W_m) - s_{\text{t1bar}_q} - \text{cc}_{\text{top}}}{s_{\text{topo}}} \right\rceil + 1 & \text{if } W_m < \frac{W_{\text{top}_q}}{2} \\ n_{\text{topm}_q} & \text{otherwise} \end{cases}$$

Bar counter:

$$it := 1, 2 \dots n_{\text{top}_{n+nj}}$$

Distance of bars from centerline across top of footing:

$$z_{\text{top}_{it}} := \text{if}\left[it \leq n_{\text{top}_{n+nj}}, it \cdot s_{\text{topm}}, s_{\text{topm}} \cdot n_{\text{top}_{n+nj}} + s_{\text{topo}} \cdot \left(it - n_{\text{top}_{n+nj}}\right)\right]$$

Depth of footing for top middle steel at point:

$$d_{\text{topm}_i} := h_b + \frac{x_{\text{alt}_i}}{a} \cdot h_c - \text{cc}_{\text{top}} - d_{i_{\text{topm}}}$$

$$d_{\text{topm}_j} := h_b + h_c - \text{cc}_{\text{top}} - d_{i_{\text{topm}}}$$

Depth of footing for top outside steel at point:

$$d_{\text{topo}_i} := h_b + \frac{x_{\text{alt}_i}}{a} \cdot h_c - \text{cc}_{\text{top}} - d_{i_{\text{topo}}}$$

$$d_{\text{topo}_j} := h_b + h_c - \text{cc}_{\text{top}} - d_{i_{\text{topo}}}$$

Depth of each bar at section:

$$d_{\text{bart}_{q,it}} := \begin{cases} \text{if } it \cdot s_{\text{topm}} > \frac{B}{2} \\ \left| \begin{array}{l} d_{\text{topm}_q} - \left(z_{\text{top}_{it}} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{if } it \leq n_{\text{topm}_q} \\ d_{\text{topo}_q} - \left(z_{\text{top}_{it}} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{otherwise} \end{array} \right. \\ d_{\text{topm}_q} \quad \text{otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{\text{stoptm}_q} := \text{if}\left(x_{\text{alt}_q} \geq \text{cd}_{\text{top}}, n_{\text{topm}_q} \cdot 2A_{\text{topm}}, \frac{1}{2} \cdot n_{\text{topm}_q} \cdot 2A_{\text{topm}}\right) + A_{\text{topm}}$$

Area of steel provided across section at outside section:

$$A_{\text{stopto}_q} := \text{if}\left[x_{\text{alt}_q} \geq \text{cd}_{\text{top}}, \left(n_{\text{top}_q} - n_{\text{topm}_q}\right) \cdot 2 \cdot A_{\text{topo}}, \frac{1}{2} \cdot \left(n_{\text{top}_q} - n_{\text{topm}_q}\right) \cdot 2 \cdot A_{\text{topo}}\right]$$

Minimum area of steel required at section:

$$A_{\text{smint}_q} := \rho_{T,\min} \cdot \text{ConcreteVolume}(x_q) \quad (\text{Reference 1a})$$

Factored moment considering minimum reinforcing requirements at section:

$$M_{utop\ q} := \text{if} \left( A_{stoptm\ q} + A_{stopo\ q} \leq A_{smint\ q}, \max \left( \frac{4}{3} \cdot M_{utop1\ q}, 0 \right), \max \left( M_{utop1\ q}, 0 \right) \right)$$

Distance from section to end of bar for continuous bars:

$$l_{top1\ q, it} := \text{if} \left[ z_{top, it} > \frac{B}{2}, \max \left[ x_{alt\ q} - \left( z_{top, it} - \frac{B}{2} \right) - \sqrt{2} \cdot cc_{top}, 0 \right], \max \left( x_{alt\ q} - cc_{top}, 0 \right) \right]$$

Distance from section to end of bar of bar for cut off bars:

$$l_{top2\ q, it} := \max \left( x_{alt\ q} - cd_{top}, 0 \right)$$

Selection of appropriate bar end distance for bar in question:

$$l_{top\ q, it} := \text{if} \left( it \cdot s_{topm} - \frac{B}{2} < cd_{top} \wedge \frac{it}{2} \neq \text{trunc} \left( \frac{it}{2} \right), l_{top2\ q, it}, l_{top1\ q, it} \right)$$

Force developed in each middle bar including development length of individual bars:

$$f_{sm\ q, it} := \text{if} \left( it \leq n_{top\ q}, A_{topm} \cdot f_{yT} \cdot \min \left( \frac{l_{top\ q, it}}{l_{dtopm}}, 1 \right), 0 \right)$$

Force developed in each outside bar including development length of individual bars:

$$f_{so\ q, it} := \text{if} \left( it \leq n_{top\ q}, A_{topo} \cdot f_{yT} \cdot \min \left( \frac{l_{top\ q, it}}{l_{dtopo}}, 1 \right), 0 \right)$$

Beta factor:

$$\beta_1 := \text{if} \left[ f_c \geq 4000 \text{psi}, \max \left[ 0.85 - 0.05 \cdot \left( \frac{\frac{f_c}{\text{psi}} - 4000}{1000} \right), 0.65 \right], 0.85 \right] \beta_1 = 0.80$$

Depth of neutral axis at section:

$$xop\ q := \frac{A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt\ q} - cc_{top}}{l_{dtopm}}, 1 \right) + 2 \cdot \sum_{kk=1}^{n_{topm\ q}} f_{sm\ q, kk} + 2 \cdot \sum_{jj=(n_{topm\ q})+1}^{n_{top\ q}} f_{so\ q, jj}}{W_{top\ q} \beta_1 \cdot 0.85 \cdot f_c}$$

Factored moment capacity at section:

$$\phi M_{ntop\ q} := \phi_b \cdot \left[ A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt\ q} - cc_{top}}{l_{dtopm}}, 1 \right) \cdot \left( d_{bart\ q, 1} - xop\ q \right) + W_{top\ q} \cdot 0.85 \cdot \beta_1 \cdot f_c \cdot xop\ q \cdot \left[ xop\ q - \left( \frac{xop\ q \cdot \beta_1}{2} \right) \right] \dots \right. \\ \left. + 2 \cdot \sum_{kk=1}^{n_{topm\ q}} \left[ f_{sm\ q, kk} \cdot \left( d_{bart\ q, kk} - xop\ q \right) \right] + 2 \cdot \sum_{jj=(n_{topm\ q})+1}^{n_{top\ q}} \left[ f_{so\ q, jj} \cdot \left( d_{bart\ q, jj} - xop\ q \right) \right] \right]$$

Check of factored moment vs. moment capacity at each section:

$$\text{check}_{top\ q} := \frac{M_{utop\ q}}{\phi M_{ntop\ q}}$$

Factored moment capacity in middle section at critical section:

$$\phi M_{nbotm_{n+n_j}} := \phi_b \cdot \left[ A_{botm} \cdot f_{yB} \cdot \min \left[ \frac{I_{bot(n+n_j),1}}{I_{dbotm}}, 1 \right] \cdot \left( d_{botm_{n+n_j}} - \frac{a_{botm}}{2} \right) + 2 \cdot \sum_{kk=1}^{n_{botm_{n+n_j}}} \left[ A_{botm} \cdot f_{yB} \cdot \min \left[ \frac{I_{bot(n+n_j),kk}}{I_{dbotm}}, 1 \right] \cdot \left[ d_{barb_{(n+n_j),kk}} - \frac{a_{botm}}{2} \right] \right] \right]$$

$$\phi M_{nbotm_{n+n_j}} = 20250 \cdot \text{kip} \cdot \text{ft}$$

Factored moment capacity in middle section at critical section:

$$\phi M_{ntopm_{n+n_j}} := \phi_b \cdot \left[ A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt_{n+n_j}} - cc_{top}}{I_{dtopm}}, 1 \right) \cdot \left( d_{bart_{n+n_j},1} - x_{op_{n+n_j}} \right) + W_m \cdot 0.85 \cdot \beta_1 \cdot f_c \cdot x_{op_{n+n_j}} \cdot \left[ x_{op_{n+n_j}} - \left( \frac{x_{op_{n+n_j}} \cdot \beta_1}{2} \right) \right] \dots \right]$$

$$+ 2 \cdot \sum_{kk=1}^{n_{topm_{n+n_j}}} \left[ f_{sm_{n+n_j},kk} \cdot \left( d_{bart_{n+n_j},kk} - x_{op_{n+n_j}} \right) \right]$$

$$\phi M_{ntopm_{n+n_j}} = 6774 \cdot \text{kip} \cdot \text{ft}$$

Unbalanced wind moment on joint:

$$M_{unbalancedW} := M_{dW}$$

$$M_{unbalancedW} = 31305 \cdot \text{kip} \cdot \text{ft}$$

Fraction of wind moment carried by flexure:

$$\gamma_{fW} := \frac{\phi M_{nbotm_{n+n_j}} + \phi M_{ntopm_{n+n_j}}}{M_{unbalancedW}}$$

$$\gamma_{fW} = 0.86$$

Unbalanced seismic moment on joint:

$$M_{unbalancedEQ} := M_{dEQ}$$

$$M_{unbalancedEQ} = 14440 \cdot \text{kip} \cdot \text{ft}$$

Fraction of seismic moment carried by flexure:

$$\gamma_{fEQ} := \frac{\phi M_{nbotm_{n+n_j}} + \phi M_{ntopm_{n+n_j}}}{M_{unbalancedEQ}}$$

$$\gamma_{fEQ} = 1.87$$



### K. Bottom Moment Capacity Results

$q =$	$x_q =$	$W_{bot\ q} =$	$A_{sboto\ q} + A_{sbotm\ q} =$	$A_{sminb\ q} =$	$M_{ubot\ q} =$	$\phi M_{nbot\ q} =$	$check_{bot\ q} =$
	ft	ft	$\cdot in^2$	$\cdot in^2$	$\cdot k \cdot ft$	$\cdot k \cdot ft$	
1	0.5	21.7	45.2	10.9	0	648	0.00
2	1.0	22.7	45.2	11.9	0	1546	0.00
3	1.5	23.7	48.4	12.8	0	2507	0.00
4	2.0	24.7	51.5	13.8	0	3581	0.00
5	2.5	25.7	51.5	14.8	35	4838	0.01
6	3.0	26.7	54.6	15.8	114	6170	0.02
7	3.5	27.7	57.7	16.8	225	7625	0.03
8	4.0	28.7	57.7	17.9	367	8903	0.04
9	4.5	29.7	60.8	18.9	541	9537	0.06
10	5.0	30.7	64.0	20.0	745	10188	0.07
11	5.5	31.7	64.0	21.1	980	10908	0.09
12	6.0	32.7	68.3	22.3	1246	11556	0.11
13	6.5	33.7	68.3	23.4	1543	12262	0.13
14	7.0	34.7	68.3	24.6	1870	12967	0.14
15	7.5	35.7	69.5	25.8	2227	13665	0.16
16	8.0	36.7	70.7	27.0	2618	14368	0.18
17	8.5	37.7	70.7	28.2	3035	15064	0.20
18	9.0	38.7	71.9	29.5	3483	15725	0.22
19	9.5	39.7	73.1	30.7	3960	16360	0.24
20	10.0	40.7	73.1	32.0	4466	16942	0.26
21	10.5	41.7	74.3	33.3	5002	17534	0.29
22	11.0	42.7	75.5	34.7	5566	18133	0.31
23	11.5	43.7	75.5	36.0	6160	18730	0.33
24	12.0	44.7	76.7	37.4	6782	19337	0.35
25	12.5	45.7	77.9	38.8	7433	19952	0.37
26	13.0	46.7	77.9	40.2	8113	20565	0.39
27	13.5	47.7	79.1	41.6	8821	21188	0.42
28	14.0	48.7	80.3	43.1	9553	21818	0.44
29	14.5	49.7	80.3	44.5	10284	22446	0.46
30	15.1	50.0	81.5	45.0	11141	22737	0.49
31	15.7	50.0	81.5	45.0	11987	22852	0.52
32	16.3	50.0	81.5	45.0	12822	22932	0.56
33	16.9	50.0	81.5	45.0	13645	22974	0.59
34	17.5	50.0	81.5	45.0	14457	22979	0.63

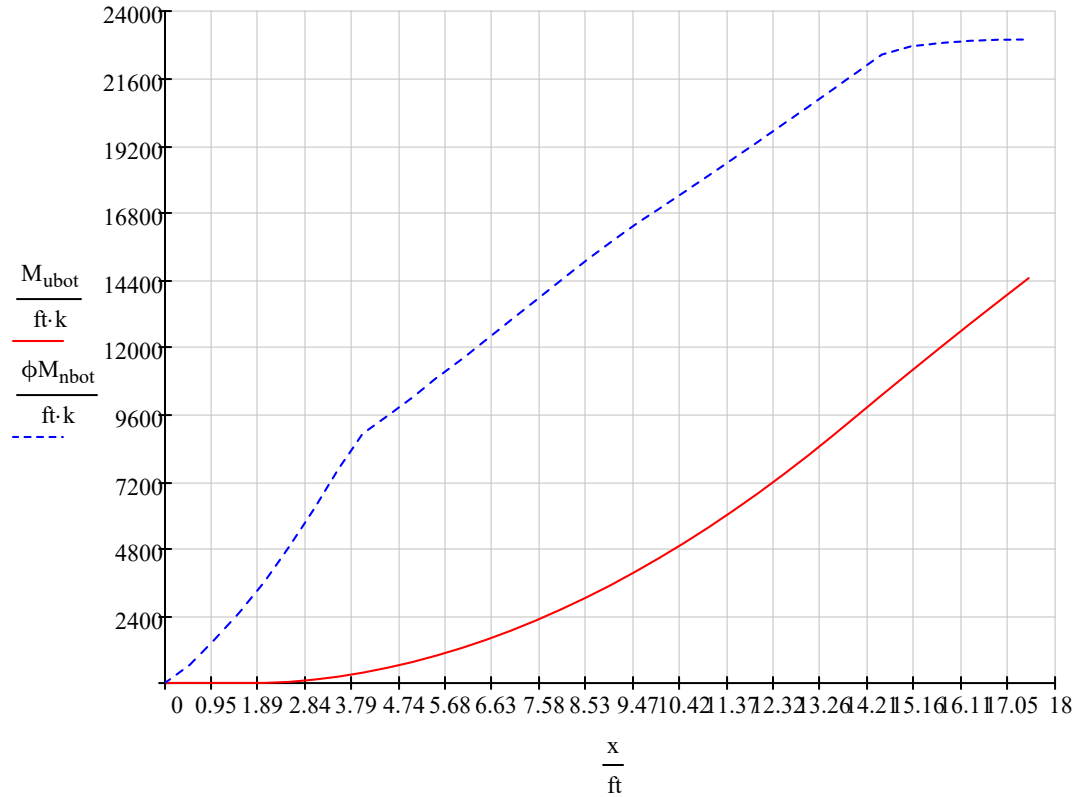
$check_{bot\ n+n_j} = 0.63$

**L. Top Moment Capacity Results**

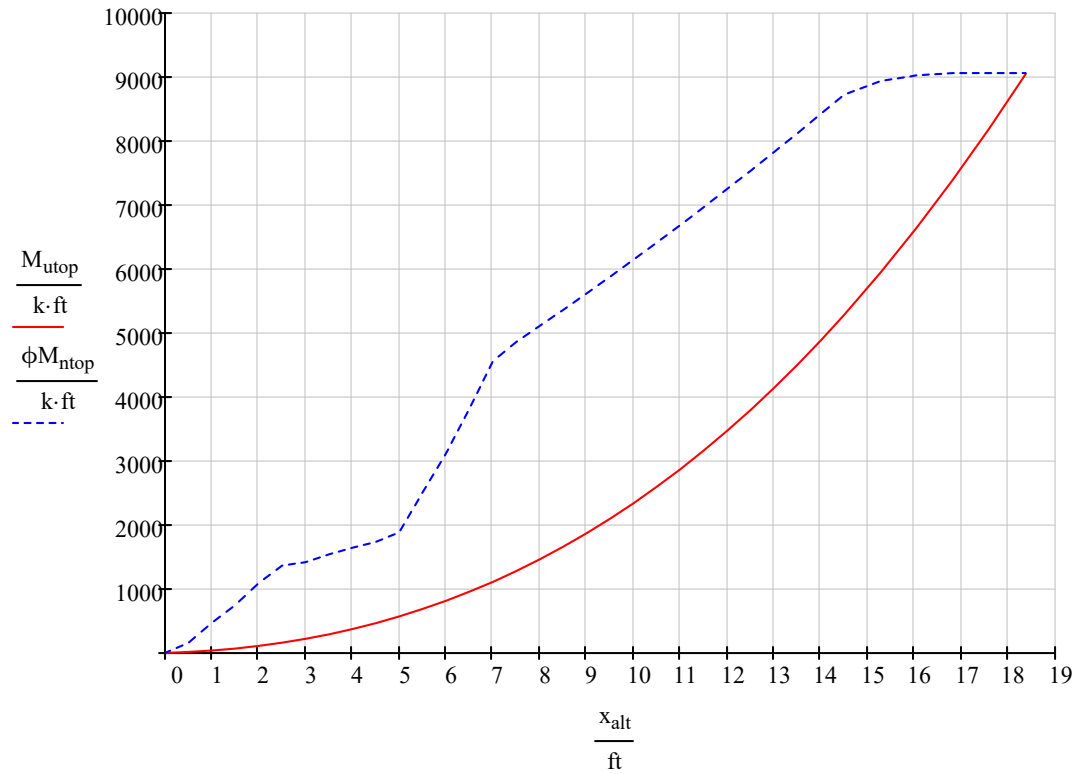
q =	$x_{alt\ q} =$ ft	$W_{top\ q} =$ ft	$A_{stopo\ q} + A_{stopm\ q} =$ $A_{smint\ q} =$ $\cdot in^2$	$M_{utop\ q} =$ $\cdot ft\cdot k$	$\phi M_{ntop\ q} =$ $\cdot ft\cdot k$	check <sub>top<sub>q</sub></sub> =
1	0.50	21.7	7.0	14	158	0.09
2	1.00	22.7	7.5	37	469	0.08
3	1.50	23.7	7.9	69	748	0.09
4	2.00	24.7	8.4	109	1093	0.10
5	2.50	25.7	8.4	160	1365	0.12
6	3.00	26.7	8.8	220	1418	0.16
7	3.50	27.7	9.2	291	1541	0.19
8	4.00	28.7	9.7	373	1643	0.23
9	4.50	29.7	9.7	466	1734	0.27
10	5.00	30.7	19.8	570	1885	0.30
11	5.50	31.7	20.7	686	2508	0.27
12	6.00	32.7	22.4	815	3119	0.26
13	6.50	33.7	22.4	957	3821	0.25
14	7.00	34.7	22.4	1111	4560	0.24
15	7.50	35.7	23.3	1279	4861	0.26
16	8.00	36.7	24.2	1462	5110	0.29
17	8.50	37.7	25.1	1659	5359	0.31
18	9.00	38.7	25.1	1870	5616	0.33
19	9.50	39.7	26.0	2097	5878	0.36
20	10.00	40.7	26.8	2339	6145	0.38
21	10.50	41.7	27.7	2596	6411	0.40
22	11.00	42.7	27.7	2871	6686	0.43
23	11.50	43.7	28.6	3162	6965	0.45
24	12.00	44.7	29.5	3470	7249	0.48
25	12.50	45.7	30.4	3796	7533	0.50
26	13.00	46.7	30.4	4139	7825	0.53
27	13.50	47.7	31.2	4501	8121	0.55
28	14.00	48.7	32.1	4882	8422	0.58
29	14.50	49.7	33.0	5282	8723	0.61
30	15.28	50.0	33.0	5941	8936	0.66
31	16.05	50.0	33.0	6647	9026	0.74
32	16.83	50.0	33.0	7400	9060	0.82
33	17.60	50.0	33.0	8200	9060	0.91
34	18.38	50.0	33.0	9047	9060	1.00

check<sub>top<sub>n+n<sub>j</sub></sub></sub> = 1.00

**Factored Moment vs.  
 Moment Capacity -  
 Bottom Reinforcing**



**Factored Moment vs.  
 Moment Capacity -  
 Top Reinforcing**



### M. Check Cutoff Locations

(Reference 1a)

Distance to top bar cutoff location from edge of footing:  $cd_{top} = 5.0 \cdot ft$

Counter corresponding to cutoff location:  $b := \frac{cd_{top}}{x_{alt_1}}$

Distance to cutoff location from edge of footing:  $x_{alt_b} = 5.0 \cdot ft$

Factored moment capacity at cutoff location:  $\phi M_{ntop_b} = 1885 \cdot k \cdot ft$

Footing effective depth at cutoff location:  $d_{topm_b} = 3.1 \cdot ft$

Distance equal to effective depth from cutoff location:  $cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm}) = 8.1 \cdot ft$

Counter corresponding to bin below distance equal to effective depth from cutoff location:  $b_{under} := \text{trunc}\left(2 \cdot \frac{cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm})}{ft}\right) \quad b_{under} = 16$

$x_{alt_{b_{under}}} = 8.0 \cdot ft$

Moment at distance equal to effective depth from cut:

$$M_{utopcutd} := M_{utop_{b_{under}}} \dots$$

$$+ \left( cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm}) - x_{alt_{b_{under}}} \right) \cdot \left[ \frac{M_{utop_{(b_{under}+1)}} - M_{utop_{b_{under}}}}{0.5ft} \right]$$

$$M_{utopcutd} = 1490 \cdot k \cdot ft$$

Check factored moment at distance equal to effective depth from cutoff location:

$$\frac{M_{utopcutd}}{\phi M_{ntop_b}} = 0.79$$

Distance to bottom bar cutoff location from edge of footing:

$$cd_{bot} = 0.0 \text{ ft}$$

Counter corresponding to cutoff location:

$$b := \frac{cd_{bot}}{x_1}$$

Distance to cutoff location from edge of footing:

$$x_b = 0.0 \text{ ft}$$

Factored moment capacity at cutoff location:

$$\phi M_{nbot_b} = 0 \cdot k \cdot ft$$

Footing effective depth at cutoff location:

$$d_{botm_b} = 0.0 \text{ ft}$$

Distance equal to effective depth from cutoff location:

$$cd_{bot} + \max(d_{botm_b}, di_{botm}) = 0.1 \text{ ft}$$

Counter corresponding to bin below distance equal to effective depth from cutoff location:

$$b_{under2} := \text{trunc} \left( 2 \cdot \frac{cd_{bot} + \max(d_{botm_b}, di_{botm})}{ft} \right) \quad b_{under2} = 0$$

$$x_{b_{under2}} = 0.0 \text{ ft}$$

Moment at distance equal to effective depth from cut:

$$M_{ubotcutd} := M_{ubot_{b_{under2}}} \dots$$

$$+ \left( cd_{bot} + \max(d_{botm_b}, di_{botm}) - x_{b_{under2}} \right) \cdot \left[ \frac{M_{ubot_{(b_{under2}+1)}} - M_{ubot_{b_{under2}}}}{0.5ft} \right]$$

$$M_{ubotcutd} = 0 \cdot k \cdot ft$$

Check factored moment at distance equal to effective depth from cutoff location:

$$\frac{M_{ubotcutd}}{\phi M_{nbot_b}} = 0.00$$

## IX-b. Moment Capacity of Bottom Reinforcement at 45 degree angle

Distance to critical section from centerline of foundation:

$$x_o := \frac{\sqrt{\pi} \cdot C}{4} \quad x_o = 90.4 \cdot \text{in}$$

Slant distance on critical section from foundation edge to slope transition point:

$$a_p := \left(\frac{1}{2}\right) \cdot \sqrt{2 \cdot \left(\frac{D - B}{2}\right)^2} - x_o \quad a_p = 33.9 \cdot \text{in}$$

Geometric distance to transition point:

$$C_1 := \sqrt{2} \cdot a_p \quad C_1 = 47.9 \cdot \text{in}$$

Height of transition point:

$$h := h_b + \left(\frac{h_c}{a}\right) \cdot C_1 \quad h = 37.3 \cdot \text{in}$$

Number of bars between centerline and critical section:

$$N_{\text{barsdown}} := \text{trunc}\left(\frac{x_o}{\sqrt{2} \cdot s_{\text{botm}}}\right) \quad N_{\text{barsdown}} = 7$$

Spacing to first bar on critical section:

$$x_1 := \sqrt{2} \cdot s_{\text{botm}} - (x_o - \sqrt{2} \cdot s_{\text{botm}} \cdot N_{\text{barsdown}}) \quad x_1 = 11.43 \cdot \text{in}$$

Number of middle section bars:

$$N_{\text{barbm}} := n_{\text{botm}_{n+n_j}} - N_{\text{barsdown}} \quad N_{\text{barbm}} = 14$$

Distance to last middle bar:

$$d_{\text{tp}} := \sqrt{2} \cdot (N_{\text{barbm}} - 1) \cdot s_{\text{botm}} + x_1 \quad d_{\text{tp}} = 177 \cdot \text{in}$$

Number of outside section bars:

$$N_{\text{barbo}} := \text{trunc}\left(\frac{\frac{D}{2} - cc_{\text{top}} - d_{\text{tp}}}{\sqrt{2} \cdot s_{\text{boto}}}\right) \quad N_{\text{barbo}} = 9$$

Number of bars crossing the critical section in diagonal direction 1:

$$N_{\text{bars1}} := N_{\text{barbm}} + N_{\text{barbo}} \quad N_{\text{bars1}} = 23$$

Bar counter for diagonal bars in diagonal direction 1:

$$b_1 := 1, 2 \dots N_{\text{bars1}}$$

Distance from centerline of bars in diagonal direction 1:

$$x_{b_1} := \text{if}\left[b_1 > N_{\text{barbm}}, x_{N_{\text{barbm}}} + \sqrt{2} \cdot s_{\text{boto}} \cdot (b_1 - N_{\text{barbm}}), x_1 + \sqrt{2} \cdot s_{\text{botm}} \cdot (b_1 - 1)\right]$$

Development lengths provided for individual bars:

$$L_{\text{in}_{b_1}} := \min\left[100 \cdot \text{in}, \sqrt{2} \cdot \left(\frac{D}{2} - cc_{\text{top}} - x_{b_1}\right)\right] \quad L_{\text{out}_{b_1}} := 100 \cdot \text{in}$$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{\text{bot}} - di_{\text{botm}} & \text{if } x_{b_1} \leq \frac{D}{2} - (a_p + 2 \cdot x_o) \\ \frac{h_b - h}{a_p} \cdot \left[x_{b_1} - \left(\frac{D}{2} - a_p\right)\right] + h - (cc_{\text{bot}} + di_{\text{botm}}) & \text{if } x_{b_1} > \frac{D}{2} - a_p \\ \frac{h - (h_b + h_c)}{2 \cdot x_o} \cdot \left[x_{b_1} - \left[\frac{D}{2} - (a_p + 2 \cdot x_o)\right]\right] + h_c + h_b - (cc_{\text{bot}} + di_{\text{botm}}) & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \text{trunc} \left[ \frac{\left( \frac{D}{2} - cc_{top} \right) - \left( \sqrt{2} \cdot s_{botm} - x_1 \right)}{\sqrt{2} \cdot s_{botm}} + 1 \right] \quad N_{bars2} = 24$$

Average width of tributary area for individual bars:

$$b := \frac{\frac{D}{2}}{N_{bars1} + N_{bars2}} \quad b = 6.38 \cdot \text{in}$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma_{1_{b1}} := \begin{cases} \frac{\sqrt{2} \cdot f_{yB} \cdot A_{boto}}{2} \cdot \min \left( \frac{\min(L_{in_{b1}}, L_{out_{b1}})}{l_{dboto}}, 1 \right) & \text{if } b_1 > N_{barbm} \\ \frac{\sqrt{2} \cdot f_{yB} \cdot A_{botm}}{2} \cdot \min \left( \frac{\min(L_{in_{b1}}, L_{out_{b1}})}{l_{dbotm}}, 1 \right) & \text{otherwise} \end{cases}$$

Depth of compression block for bars in diagonal direction 1:

$$a_{1_{b1}} := \frac{\sigma_{1_{b1}}}{0.85 \cdot f_c \cdot b}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 \dots N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2} \cdot s_{botm} \cdot (b_2) - 2 \cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$\text{Centerbar} := N_{barsdown} + 1 \quad \text{Centerbar} = 8$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \frac{\text{Centerbar}}{2} > \text{trunc} \left( \frac{\text{Centerbar}}{2} \right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$\text{Other}_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \quad \text{Other}_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} \text{Other}_{bars} & \text{if } \frac{b_2}{2} > \text{trunc} \left( \frac{b_2}{2} \right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b_2}} := 100 \text{in}$$

$L_{out2_{b_2}} :=$  if  $C_{bar_{b_2}} = \text{"noncutoff"}$

$$\begin{cases} \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left( \frac{D}{2} - cc_{top} - \lambda_{b_2} \right) \right] & \text{if } \frac{D}{2} - \lambda_{b_2} < \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \\ \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b_2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] \right] \right] & \text{otherwise} \\ \max \left[ 0 \text{in}, \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b_2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] \right] - (cd_{bot} - cc_{top}) \right] \right] & \text{otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b2} := \begin{cases} h_b + h_c - cc_{bot} - di_{botm} & \text{if } \lambda_{b2} \leq \frac{D}{2} - (a_p + 2 \cdot x_o) \\ \frac{h_b - h}{a_p} \cdot \left[ \lambda_{b2} - \left( \frac{D}{2} - a_p \right) \right] + h - (cc_{bot} + di_{botm}) & \text{if } \lambda_{b2} > \frac{D}{2} - a_p \\ \frac{h - (h_b + h_c)}{2 \cdot x_o} \cdot \left[ \lambda_{b2} - \left[ \frac{D}{2} - (a_p + 2 \cdot x_o) \right] \right] + h_c + h_b - (cc_{bot} + di_{botm}) & \text{otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma2_{b2} := \frac{\sqrt{2} \cdot f_{yB} \cdot A_{botm}}{2} \cdot \min \left( \frac{\min(L_{in2_{b2}}, L_{out2_{b2}})}{l_{dbotm}}, 1 \right)$$

Depth of compression block for bars in diagonal direction 2:

$$a2_{b2} := \frac{\sigma2_{b2}}{0.85 \cdot f_c \cdot b}$$

Factored moment capacity at section:

$$\phi M_{nbot} := \phi_b \cdot \left[ 2 \cdot \left[ \sum_{kk=1}^{N_{bars1}} \left[ \sigma1_{kk} \cdot \left( d_{kk} - \frac{a1_{kk}}{2} \right) \right] \right] + 2 \cdot \left[ \sum_{kk=1}^{N_{bars2}} \left[ \sigma2_{kk} \cdot \left( d2_{kk} - \frac{a2_{kk}}{2} \right) \right] \right] \right] \dots \phi M_{nbot} = 23730 \cdot k \cdot ft$$

Ultimate moment in bottom reinforcement at critical section:

$$M_{ubot_{n+nj}} = 14457 \cdot k \cdot ft$$

Check of factored moment against moment capacity at critical section:

$$check_{bot} := \frac{M_{ubot_{n+nj}}}{\phi M_{nbot}} \quad \boxed{check_{bot} = 0.61}$$



### IX-c. Moment Capacity of Top Reinforcement at 45 degree angle

Distance to alternate critical section from centerline of foundation:

$$x_{alt.o} := \frac{\sqrt{\pi} \cdot OD}{4} \quad x_{alt.o} = 79.5 \cdot \text{in}$$

Slant distance on alternate critical section from foundation edge to slope transition point:

$$a_{alt.p} := \left(\frac{1}{2}\right) \cdot \sqrt{2 \cdot \left(\frac{D-B}{2}\right)^2} - x_{alt.o} \quad a_{alt.p} = 44.8 \cdot \text{in}$$

Geometric distance to transition point:

$$C_{alt.1} := \sqrt{2} \cdot a_{alt.p} \quad C_{alt.1} = 63.3 \cdot \text{in}$$

Height of transition point:

$$h_{alt} := h_b + \left(\frac{h_c}{a}\right) \cdot C_{alt.1} \quad h_{alt} = 40.3 \cdot \text{in}$$

Vertical distance to the centerline of top bars in outer section:

$$K_1 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h_{alt} - h_b}{a_{alt.p}}\right)\right)} \quad K_1 = 2.86 \cdot \text{in}$$

Vertical distance to the centerline of top bars in middle section:

$$K_2 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h_b + h_c - h_{alt}}{2 \cdot x_{alt.o}}\right)\right)} \quad K_2 = 2.78 \cdot \text{in}$$

Number of bars between centerline and critical section:

$$N_{barsdown} := \text{trunc}\left(\frac{x_{alt.o}}{\sqrt{2} \cdot s_{topm}}\right) \quad N_{barsdown} = 7$$

Spacing to first bar on critical section:

$$x_1 := \sqrt{2} \cdot s_{topm} - (x_{alt.o} - \sqrt{2} \cdot s_{topm} \cdot N_{barsdown}) \quad x_1 = 11.03 \cdot \text{in}$$

Number of middle section bars:

$$N_{bartm} := n_{topm_{n+n_j}} - N_{barsdown} \quad N_{bartm} = 17$$

Distance to last middle bar:

$$d_{tp} := \sqrt{2} \cdot (N_{bartm} - 1) \cdot s_{topm} + x_1 \quad d_{tp} = 192 \cdot \text{in}$$

Number of outside section bars:

$$N_{barto} := \text{trunc}\left(\frac{\left(\frac{D}{2} - cc_{top} - d_{tp}\right)}{\sqrt{2} \cdot s_{topo}}\right) \quad N_{barto} = 9$$

Number of bars crossing the critical section in diagonal direction 1:

$$N_{bars1} := N_{bartm} + N_{barto} \quad N_{bars1} = 26$$

Bar counter for diagonal bars in diagonal direction 1:

$$b_1 := 1, 2 \dots N_{bars1}$$

Distance from centerline of bars in diagonal direction 1:

$$x_{b_1} := \text{if}\left[b_1 > N_{bartm}, x_{N_{bartm}} + \sqrt{2} \cdot s_{topo} \cdot (b_1 - N_{bartm}), x_1 + \sqrt{2} \cdot s_{topm} \cdot (b_1 - 1)\right]$$

Development lengths provided for individual bars:

$$L_{in_{b_1}} := \min\left[100 \cdot \text{in}, \sqrt{2} \cdot \left(\frac{D}{2} - cc_{top} - x_{b_1}\right)\right] \quad L_{out_{b_1}} := 100 \cdot \text{in}$$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{top} - di_{topm} & \text{if } x_{b_1} \leq \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \\ \frac{h_b - h_{alt}}{a_{alt.p}} \cdot \left[ x_{b_1} - \left( \frac{D}{2} - a_{alt.p} \right) \right] + h_{alt} - K_1 & \text{if } x_{b_1} > \frac{D}{2} - a_{alt.p} \\ \frac{h_{alt} - (h_b + h_c)}{2 \cdot x_{alt.o}} \cdot \left[ x_{b_1} - \left[ \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \right] \right] + h_c + h_b - K_2 & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \text{trunc} \left[ \frac{\left( \frac{D}{2} - cc_{top} \right) - \left( \sqrt{2} \cdot s_{topm} - x_1 \right)}{\sqrt{2} \cdot s_{topm}} + 1 \right] \quad N_{bars2} = 27$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma_{l_{b_1}} := \begin{cases} \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topo}}{2} \cdot \min \left( \frac{\min(L_{in_{b_1}}, L_{out_{b_1}})}{l_{dtopo}}, 1 \right) & \text{if } b_1 > N_{bartm} \\ \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topm}}{2} \cdot \min \left( \frac{\min(L_{in_{b_1}}, L_{out_{b_1}})}{l_{dtopm}}, 1 \right) & \text{otherwise} \end{cases}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 \dots N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2} \cdot s_{topm} \cdot (b_2) - 2 \cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$\text{Centerbar} := N_{barsdown} + 1 \quad \text{Centerbar} = 8$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \frac{\text{Centerbar}}{2} > \text{trunc} \left( \frac{\text{Centerbar}}{2} \right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$\text{Other}_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \quad \text{Other}_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} \text{Other}_{bars} & \text{if } \frac{b_2}{2} > \text{trunc} \left( \frac{b_2}{2} \right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b2}} := 100\text{in}$$

$$L_{out2_{b2}} := \begin{cases} \text{if } C_{bar_{b2}} = \text{"noncutoff"} \\ \left[ \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left( \frac{D}{2} - cc_{top} - \lambda_{b2} \right) \right] \text{ if } \frac{D}{2} - \lambda_{b2} < \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right. \\ \left. \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] \right] \right] \right] \text{ otherwise} \\ \max \left[ 0 \text{in}, \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] \right] - (cd_{top} - cc_{top}) \right] \right] \text{ otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b2} := \begin{cases} h_b + h_c - cc_{top} - d_{i_{topm}} \text{ if } \lambda_{b2} \leq \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \\ \frac{h_b - h_{alt}}{a_{alt.p}} \cdot \left[ \lambda_{b2} - \left( \frac{D}{2} - a_{alt.p} \right) \right] + h_{alt} - K_1 \text{ if } \lambda_{b2} > \frac{D}{2} - a_{alt.p} \\ \frac{h_{alt} - (h_b + h_c)}{2 \cdot x_{alt.o}} \cdot \left[ \lambda_{b2} - \left[ \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \right] \right] + h_c + h_b - K_2 \text{ otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma_{2_{b2}} := \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topm}}{2} \cdot \min \left( \frac{\min(L_{in2_{b2}}, L_{out2_{b2}})}{l_{dtopm}}, 1 \right)$$

Footing is separated into strips containing one bar each. Depth of compression block for each strip:

$$a_{top} := \frac{\sum_{ii=1}^{N_{bars1}} \sigma_{1_{ii}} + \sum_{jj=1}^{N_{bars2}} \sigma_{2_{jj}}}{0.85 \cdot f_c \cdot \frac{D}{2}} \quad a_{top} = 0.90 \cdot \text{in}$$

Factored moment capacity at critical section:

$$\phi M_{ntop} := \phi_b \cdot \left[ 2 \cdot \left[ \sum_{kk=1}^{N_{bars1}} \left[ \sigma_{1_{kk}} \cdot \left( d_{kk} - \frac{a_{top}}{2} \right) \right] \right] \dots \right. \\ \left. + 2 \cdot \left[ \sum_{kk=1}^{N_{bars2}} \left[ \sigma_{2_{kk}} \cdot \left( d2_{kk} - \frac{a_{top}}{2} \right) \right] \right] \right] \quad \phi M_{ntop} = 8662 \cdot \text{k} \cdot \text{ft}$$

Ultimate moment in top reinforcement:

$$M_{utop_{n+nj}} = 9047 \cdot \text{k} \cdot \text{ft}$$

Check of factored moment against moment capacity at critical section:

$$check_{top} := \frac{M_{utop_{n+nj}}}{\phi M_{ntop}} \quad \boxed{check_{top} = 1.04}$$

### IX-d. One-Way Shear Capacity Check

Plan location of section:  $x_i := \frac{i}{2} \cdot \text{ft}$        $x_j := \max(x) + (x_{\text{face}} - \max(x)) \cdot \frac{j - n}{n_j}$

Depth, d, as a function of distance along the sloped portion of the foundation is:  $d_i := h_b + \frac{x_i}{a} \cdot h_c - cc_{\text{bot}} - \frac{d_{\text{botm}}}{2}$

$$d_j := h_b + h_c - cc_{\text{bot}} - \frac{d_{\text{botm}}}{2}$$

Determine controlling load case:  $V_{u_q} := \text{if} \left( |V_{uW_q}| \geq |V_{uEQ_q}|, V_{uW_q}, V_{uEQ_q} \right)$

Location of critical section from edge of footing:  $x_{\text{critical}} := x_{\text{face}} - d_{n+n_j}$        $x_{\text{critical}} = 12.6 \text{ ft}$  (Reference 1a)

Array counter for all slices up to the critical section:  $\text{qcs} := 1, 2 \dots \text{trunc} \left( \frac{x_{\text{critical}}}{0.5 \text{ ft}} \right) + 1$

Shear capacity between edge and a:  $\phi V_{n_i} := \phi_v \cdot 2 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[ B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right) \right]$  (Reference 1a)

Shear capacity between a and  $x_{\text{face}}$ :  $\phi V_{n_j} := \phi_v \cdot 2 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[ B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right) \right]$  (Reference 1a)

Reinforcing ratio at section between edge and a:  $\rho_{x_i} := \frac{A_{\text{sboto}_i} + A_{\text{sbotm}_i}}{B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right)}$

Reinforcing ratio at section between a and edge:  $\rho_{x_j} := \frac{A_{\text{sboto}_j} + A_{\text{sbotm}_j}}{B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right)}$

Capacity between edge and a:  $\phi V_{n2_i} := \phi_v \cdot \left( 8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \right) \cdot (\rho_{x_i})^{\frac{1}{3}} \cdot \left[ B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right) \right]$

Capacity between a and  $x_{\text{face}}$ :  $\phi V_{n2_j} := \phi_v \cdot \left( 8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \right) \cdot (\rho_{x_j})^{\frac{1}{3}} \cdot \left[ B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right) \right]$

Shear Design Results

qcs =	$x_{qcs} =$ ft	$d_{qcs} =$ in	$V_{u_{qcs}} =$ ·k	$\phi V_{n_{qcs}} =$ ·k	$\phi V_{n2_{qcs}} =$ ·k	$\frac{V_{u_{qcs}}}{\phi V_{n_{qcs}}} =$	$\frac{V_{u_{qcs}}}{\phi V_{n2_{qcs}}} =$
1	0.5	25.5	-18	703	533	-0.03	-0.03
2	1.0	26.6	-25	766	565	-0.03	-0.04
3	1.5	27.8	-32	832	610	-0.04	-0.05
4	2.0	28.9	63	898	656	0.07	0.10
5	2.5	30.1	127	967	689	0.13	0.18
6	3.0	31.3	190	1036	736	0.18	0.26
7	3.5	32.4	253	1107	783	0.23	0.32
8	4.0	33.6	316	1180	817	0.27	0.39
9	4.5	34.7	378	1254	866	0.30	0.44
10	5.0	35.9	440	1330	916	0.33	0.48
11	5.5	37.1	501	1407	951	0.36	0.53
12	6.0	38.2	563	1485	1007	0.38	0.56
13	6.5	39.4	624	1565	1043	0.40	0.60
14	7.0	40.5	684	1647	1079	0.42	0.63
15	7.5	41.7	745	1729	1122	0.43	0.66
16	8.0	42.9	805	1814	1164	0.44	0.69
17	8.5	44.0	865	1900	1201	0.46	0.72
18	9.0	45.2	924	1987	1244	0.47	0.74
19	9.5	46.4	983	2076	1288	0.47	0.76
20	10.0	47.5	1042	2166	1325	0.48	0.79
21	10.5	48.7	1100	2258	1370	0.49	0.80
22	11.0	49.8	1158	2351	1415	0.49	0.82
23	11.5	51.0	1216	2446	1453	0.50	0.84
24	12.0	52.2	1273	2542	1498	0.50	0.85
25	12.5	53.3	1330	2640	1545	0.50	0.86
26	13.0	54.5	1387	2739	1583	0.51	0.88

$$ii := 1 .. \text{trunc}\left(\frac{x_{\text{critical}}}{0.5\text{ft}}\right) + 1$$

$$\text{vals}_{ii} := \frac{V_{u_{ii}}}{\phi V_{n_{ii}}}$$

$$\text{Check}_{\text{Shear1}} := \max(\text{vals}) = 0.51$$

$$\text{Output}_{\text{ShearCriticalSection1}} := \frac{V_u}{\phi V_n} \bigg|_{\text{trunc}\left(\frac{x_{\text{critical}}}{0.5\text{ft}}\right)+1}$$

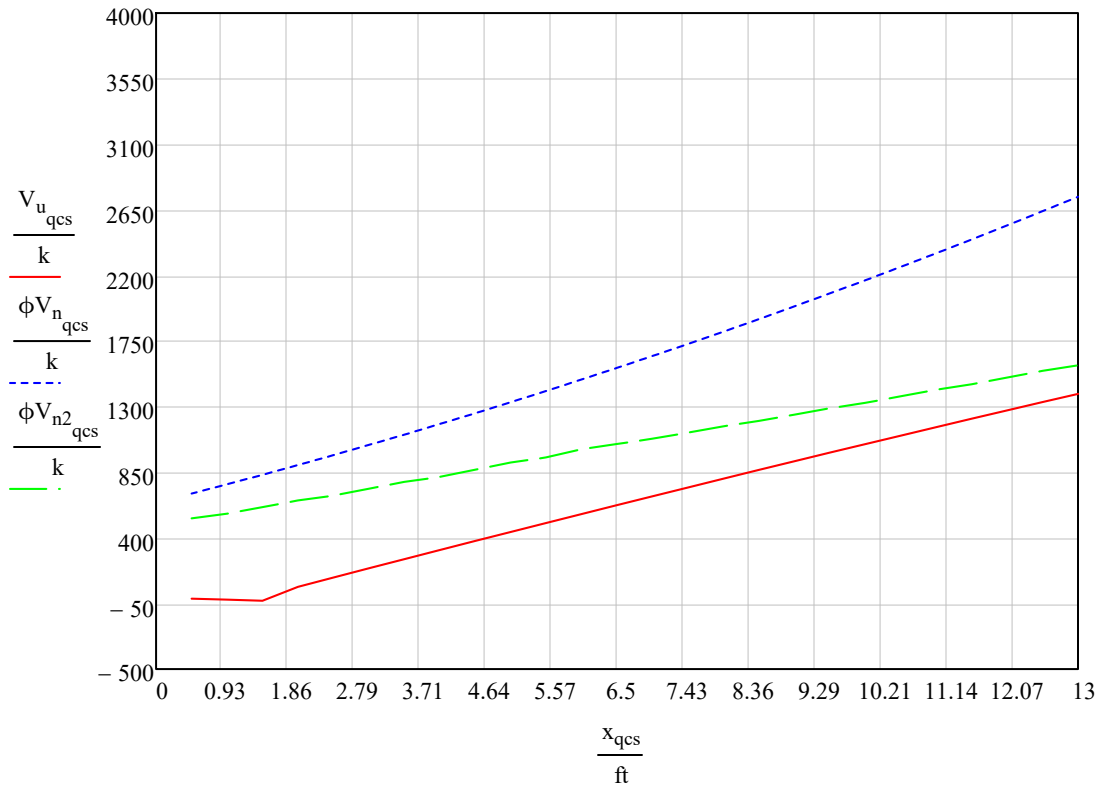
$$\text{vals}_{ii} := \frac{V_{u_{ii}}}{\phi V_{n2_{ii}}}$$

$$\text{Check}_{\text{Shear2}} := \max(\text{vals}) = 0.51$$

$$\text{Output}_{\text{ShearCriticalSection2}} := \frac{V_u}{\phi V_{n2}} \bigg|_{\text{trunc}\left(\frac{x_{\text{critical}}}{0.5\text{ft}}\right)+1}$$

**Factored Shear vs.  
 Shear Capacity -**

$x_{critical} = 12.6 \text{ ft}$



### IX-e. Pedestal Two-Way Shear Capacity Check

Effective depth at face of pedestal

$$d_{\text{mid}} := h_b + h_c - cc_{\text{bot}} - di_{\text{botm}} \quad d_{\text{mid}} = 57.6 \cdot \text{in}$$

Effective depth at face of pedestal

$$d_{\text{face}} := \text{if} \left[ \frac{C}{2} + \frac{d_{\text{mid}}}{2} < \frac{B}{2}, d_{\text{mid}} \cdot h_b + \frac{\frac{D}{2} - \left( \frac{C}{2} + \frac{d_{\text{mid}}}{2} \right)}{a} \cdot h_c - cc_{\text{bot}} - di_{\text{botm}} \right]$$

$$d_{\text{face}} = 56.3 \cdot \text{in}$$

Area of critical section:

$$A_c := 2\pi \cdot d_{\text{face}} \cdot \left( \frac{C + d_{\text{face}}}{2} \right) \quad A_c = 46066 \cdot \text{in}^2$$

Polar moment of inertia of critical section:

$$J_c := \pi \cdot d_{\text{face}} \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^3 + \left( \frac{d_{\text{face}}^3}{3} \right) \left( \frac{C + d_{\text{face}}}{2} \right) \quad J_c = 3.98 \times 10^8 \cdot \text{in}^4$$

Perimeter of critical section:

$$b_0 := 2\pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right) \quad b_0 = 818 \cdot \text{in}$$

Half critical section width:

$$c := \frac{C + d_{\text{face}}}{2} \quad (\text{interior column}) \quad c = 130 \cdot \text{in}$$

Weight of pedestal:

$$W_p = 79 \cdot \text{kip}$$

Unfactored vertical wind load on critical section:

$$P_W := W_t + W_p + (h_c + h_b) \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 \right] \cdot \gamma_c \dots$$

$$+ \gamma_{\text{sdtop}} \cdot [h_s - (h_c + h_b)] \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 - \pi \cdot \left( \frac{C}{2} \right)^2 \right]$$

$$P_W = 851 \cdot \text{k}$$

Unfactored vertical seismic load on critical section:

$$P_{EQ} := W_{OE} + W_p + (h_c + h_b) \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 \right] \cdot \gamma_c \dots$$

$$+ \gamma_{\text{sdtop}} \cdot [h_s - (h_c + h_b)] \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 - \pi \cdot \left( \frac{C}{2} \right)^2 \right]$$

$$P_{EQ} = 856 \cdot \text{k}$$

Unbalanced wind moment on joint:

$$M_{\text{unbalanced}W} = 31305 \cdot \text{kip} \cdot \text{ft}$$

Fraction of wind moment that can be carried by flexure:

$$\gamma_{fW} = 0.86$$

Fraction of wind moment carried by shear:

$$\gamma_{vW} := \max(0.4, 1 - \gamma_{fW}) \quad \gamma_{vW} = 0.40$$

Factored shear stress due to wind load at critical section:

$$v_{uW} := \frac{\alpha_{d2} \cdot P_W}{A_c} + \frac{\gamma_{vW} \cdot M_{\text{unbalanced}W} \cdot c}{J_c} \quad v_{uW} = 71 \cdot \text{psi}$$

Unbalanced seismic moment on joint:  $M_{\text{unbalancedEQ}} = 14440 \cdot \text{kip} \cdot \text{ft}$

Fraction of seismic moment that can be carried by flexure:  $\gamma_{\text{fEQ}} = 1.87$

Fraction of seismic moment carried by shear:  $\gamma_{\text{vEQ}} := \max(0.4, 1 - \gamma_{\text{fEQ}})$   $\gamma_{\text{vEQ}} = 0.40$

Factored shear stress due to seismic load at critical section:  $v_{\text{uEQ}} := \frac{\alpha_{\text{d2EQ}} \cdot P_{\text{EQ}}}{A_{\text{c}}} + \frac{\gamma_{\text{vEQ}} \cdot M_{\text{unbalancedEQ}} \cdot c}{J_{\text{c}}}$   $v_{\text{uEQ}} = 45 \cdot \text{psi}$

Determine controlling load case:  $v_{\text{u}} := \max(v_{\text{uW}}, v_{\text{uEQ}})$   $v_{\text{u}} = 71 \text{ psi}$

$$\beta_{\text{c}} := 1$$

$$\alpha_{\text{s}} := 40$$

$$\phi_{\text{v}} = 0.75$$

Shear stress capacity:  $\phi v_{\text{c}} := \phi_{\text{v}} \cdot \min \left[ \left( 2 + \frac{4}{\beta_{\text{c}}} \right), \left( \frac{\alpha_{\text{s}} \cdot d_{\text{face}}}{b_0} + 2 \right), 4 \right] \cdot \sqrt{f_{\text{c}}} \cdot \text{psi}$   $\phi v_{\text{c}} = 212 \cdot \text{psi}$  (Reference 1a)

Check of factored shear stress vs. shear stress capacity:  $\frac{v_{\text{u}}}{\phi v_{\text{c}}} = 0.34$



## X. Concrete Design - Fatigue Loads

### A. Design Functions

Function describing the volume of concrete for each slice of the moment/shear calculations.

$$\text{ConcreteVolumeFat}(y) := \begin{cases} h_b \cdot (B + 2 \cdot y) + \frac{y}{a} \cdot h_c \cdot (B + y) & \text{if } y \leq a \\ h_b \cdot (D) + h_c \cdot (B + a) & \text{otherwise} \end{cases}$$

Functions describing the weight of the soil wedge pieces acting on each slice of the moment/shear calculations.

$$\text{StaticSoilWedgeWeightFat}(\gamma_{sd}, \gamma_{ss}) := \begin{cases} \gamma_{sd} \cdot \frac{B \cdot \tan(\theta_{fat})}{2} \cdot (h_s - h_b)^2 & \text{if } d_{GWTF} \geq h_s - h_b \\ \frac{B \cdot \tan(\theta_{fat})}{2} \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{otherwise} \end{cases}$$

$$\text{VariableSoilWedgeWeightFat}(y, \gamma_{sd}, \gamma_{ss}) := \begin{cases} 0 & \text{if } d_{GWTF} \geq h_s - h_b \\ \text{otherwise} \\ \begin{cases} \sqrt{2} \cdot \tan(\theta_{fat}) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{if } y \leq a \\ \tan(\theta_{fat}) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{otherwise} \end{cases} \end{cases}$$

Function describing the volume of dry soil over each slice of the moment/shear calculations.

$$\text{DrySoilVolumeFat}(h_j, y) := \begin{cases} \text{if } d_{\text{GWTF}} \geq h_s - h_b \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \frac{y^2 \cdot h_c}{a} + \sqrt{2} \cdot \tan(\theta_{\text{fat}}) \cdot (h_s - h_b)^2 \right] & \text{if } y \leq a \\ D \cdot \left[ (h_s - h_b) - h_c \right] + h_c \cdot a + \tan(\theta_{\text{fat}}) \cdot (h_s - h_b)^2 & \text{otherwise} \\ \text{if } d_{\text{GWTF}} \leq h_s - h_j \\ d_{\text{GWTF}} \cdot (B + 2 \cdot y) & \text{if } y \leq a \\ D \cdot d_{\text{GWTF}} & \text{otherwise} \\ \text{otherwise} \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \left[ \frac{y^2 \cdot h_c}{a} - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] & \text{if } y \leq a \\ \left[ D \cdot (h_s - h_b - h_c) + \left[ h_c \cdot a - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] & \text{otherwise} \end{cases}$$

Function describing the volume of saturated soil over each slice of the moment / shear calculations.

$$\text{SaturatedSoilVolumeFat}(h_j, y) := \begin{cases} 0 & \text{if } d_{\text{GWTF}} \geq h_s - h_b \\ \text{if } d_{\text{GWTF}} \leq h_s - h_j \\ \left( (B + 2 \cdot y) \cdot \left( h_s - h_b - \frac{y}{a} \cdot h_c - d_{\text{GWTF}} \right) + \frac{y^2 \cdot h_c}{a} \right) & \text{if } y \leq a \\ (h_s - h_b - h_c - d_{\text{GWTF}}) \cdot D + h_c \cdot a & \text{otherwise} \\ \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 & \text{otherwise} \end{cases}$$

Function describing the effect of groundwater on the material weights over each slice of the moment/ shear calculations.

$$\text{BuoyancyWeightFat}(y) := \begin{cases} 0 & \text{if } d_{\text{GWTF}} \geq h_s \\ \text{if } d_{\text{GWTF}} < h_s \\ (B + 2 \cdot y) \cdot (h_s - d_{\text{GWTF}}) & \text{if } y \leq a \\ (h_s - d_{\text{GWTF}}) \cdot D & \text{otherwise} \end{cases}$$

## B. Bottom Reinforcement

(Reference 7)

Depth to reinforcement at critical section for flat portion of footing:

$$d_{\text{face2}} := h_b + h_c - cc_{\text{bot}} - 1.5d_{\text{ibotm}} - 1 \text{ in} \quad d_{\text{face2}} = 55.9 \text{ in}$$

Width of Concrete Resisting Fatigue:

$$w_f := OD + 3 \cdot (h_c + h_b)$$

$$\frac{w_f}{D} = 61\%$$

Cap width to be within "middle" strip:

$$w_f := \min(w_f, 2 \cdot W_m) \quad w_f = 30.45 \text{ ft}$$

Area of steel across critical section:

$$A_s := \frac{w_f}{s_{\text{botm}}} \cdot A_{\text{botm}} \quad A_s = 63.3 \cdot \text{in}^2$$

Footing Depth at Edge of Section:

$$d_{\text{edge}} := d_{\text{face2}} - \frac{w_f - B}{2} \cdot \frac{h_c}{a} = 44.58 \text{ in}$$

Concrete Area Resisting Shear:

$$A_{\text{face}} := B \cdot d_{\text{face2}} + 2 \cdot \frac{w_f - B}{2} \cdot \left( \frac{d_{\text{edge}} + d_{\text{face2}}}{2} \right) \quad A_{\text{face}} = 19758 \cdot \text{in}^2$$

## C. Fatigue Soil Bearing Pressure

Service load eccentricity:

$$e_{f\text{North}} := \frac{M_{\text{Unique}}{}_{\text{north}}}{W_{\text{fat}}}$$

Circular radius of octagon:

$$R := \frac{D}{2} \quad R = 25.00 \text{ ft}$$

Effective soil area in bearing:

$$A_{\text{effNorth}} := 2 \cdot \left[ \left( R^2 \right) \cdot \arccos \left( \frac{e_{f\text{North}}}{R} \right) \dots \right. \\ \left. + -e_{f\text{North}} \cdot \sqrt{R^2 - \left( e_{f\text{North}} \right)^2} \right]$$

Ellipse soil width in bearing:

$$b_{e\text{North}} := 2 \cdot \left( R - e_{f\text{North}} \right)$$

Ellipse soil length in bearing:

$$l_{e\text{North}} := 2 \cdot R \cdot \sqrt{1 - \left( 1 - \frac{b_{e\text{North}}}{2 \cdot R} \right)^2}$$

Effective soil length in bearing:

$$l_{\text{effNorth}} := \sqrt{A_{\text{effNorth}} \cdot \frac{l_{e\text{North}}}{b_{e\text{North}}}}$$

Effective soil width in bearing:

$$b_{\text{effNorth}_{q_{\text{rnorth}}}} := \frac{I_{\text{effNorth}_{q_{\text{rnorth}}}}}{I_{\text{eNorth}_{q_{\text{rnorth}}}}} \cdot b_{\text{eNorth}_{q_{\text{rnorth}}}}$$

Maximum fatigue bearing pressure:

$$f_{\text{fNorth}_{q_{\text{rnorth}}}} := \frac{W_{\text{fat}}}{A_{\text{effNorth}_{q_{\text{rnorth}}}}}$$

$$x_{\text{startNorth}_{q_{\text{rnorth}}}} := \frac{D}{2} - e_{\text{fNorth}_{q_{\text{rnorth}}}} - \frac{b_{\text{effNorth}_{q_{\text{rnorth}}}}}{2}$$

Foundation plan area:

$$A_{\text{base}} := D^2 - 2 \cdot \left( \frac{D - B}{2} \right)^2 \quad A_{\text{base}} = 2071 \text{ ft}^2$$

Section modulus of foundation for normal orientation:

$$S_{\text{normal}} := \frac{2I_{\text{fdn}}}{D} \quad S_{\text{normal}} = 13684 \cdot \text{ft}^3$$

$$W_{\text{fat}} = 1926 \cdot \text{kip}$$

$$\frac{W_{\text{fat}}}{A_{\text{base}}} = 930 \cdot \text{psf}$$

Moment at which the foundation lifts:

$$M_{\text{maxlift}} := \frac{W_{\text{fat}}}{A_{\text{base}}} \cdot S_{\text{normal}} = 12725 \cdot \text{k} \cdot \text{ft}$$

Maximum soil pressure at point when the foundation lifts:

$$\sigma_{\text{maxlift}} := \frac{W_{\text{fat}}}{A_{\text{base}}} + \frac{M_{\text{maxlift}}}{S_{\text{normal}}} = 1860 \cdot \text{psf}$$

**Min and Max soil bearing pressure for each fatigue range:**

Maximum soil pressure at point when the foundation lifts defined for each individual fatigue load:

$$\sigma_{\text{north\_max\_soiltrap}}_{qf_{\text{north}}} := \frac{W_{\text{fat}}}{A_{\text{base}}} + \frac{M_{\text{Unique}}_{\text{north}}}{S_{\text{normal}}}$$

$$\sigma_{\text{north\_min\_soil}}_{qf_{\text{north}}} := \begin{cases} \frac{W_{\text{fat}}}{A_{\text{base}}} - \frac{M_{\text{Unique}}_{\text{north}}}{S_{\text{normal}}} & \text{if } \frac{W_{\text{fat}}}{A_{\text{base}}} - \frac{M_{\text{Unique}}_{\text{north}}}{S_{\text{normal}}} > 0 \\ \text{"LIFT"} & \text{otherwise} \end{cases}$$

Guess for solver of soil bearing length:  $L_b := \frac{3}{4} \cdot D$        $L_b = 37.50 \text{ ft}$

Guess for solver of max soil pressure:  $f_{\text{max}} := \sigma_{\text{maxlift}}$

The following functions solve for the soil pressure assuming the pressure distribution is triangular and lift-off has occurred on the minimum pressure side of the foundation:

$F_{\text{VALS}}_{qf_{\text{north}}} := W_{\text{fat}}$

$M_{\text{TOEVALS}}_{qf_{\text{north}}} := W_{\text{fat}} \cdot \frac{D}{2} - M_{\text{Unique}}_{\text{north}}_{qf_{\text{north}}}$

Given

$$F = \int_0^a (B + 2 \cdot y) \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy \dots$$

$$+ \int_a^{a+B} D \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy \dots$$

$$+ \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy$$

$$M_{\text{toe}} = \int_0^a (B + 2 \cdot y) \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy \dots$$

$$+ \int_a^{a+B} D \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy \dots$$

$$+ \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy$$

$$\text{FUNCTION}(F, M_{\text{toe}}) := \text{Find}\left(\frac{L_b}{\text{ft}}, \frac{f_{\text{max}}}{\text{psf}}\right)$$

Solve the loop for the "LIFT" condition:

$$\text{Map}L_{b_{q_{r_{north}}}} := \begin{cases} \text{FUNCTION}(F_{\text{VALS}}_{q_{r_{north}}}, M_{\text{TOEVALS}}_{q_{r_{north}}}) \cdot \text{ft} & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"} \\ 0 & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} \neq \text{"LIFT"} \end{cases}$$

$$\sigma_{\text{north\_max\_soiltri}}_{q_{r_{north}}} := \begin{cases} \text{FUNCTION}(F_{\text{VALS}}_{q_{r_{north}}}, M_{\text{TOEVALS}}_{q_{r_{north}}}) \cdot \text{psf} & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"} \\ 0 & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} \neq \text{"LIFT"} \end{cases}$$

Select the actual pressure depending on if the soil pressure at each fatigue load depending on if "lift" has occurred (triangular soil pressure distribution) or not:

$$\sigma_{\text{north\_max\_soil}}_{q_{r_{north}}} := \text{if}(\sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"}, \sigma_{\text{north\_max\_soiltri}}_{q_{r_{north}}}, \sigma_{\text{north\_max\_soiltrap}}_{q_{r_{north}}}) = \dots$$

Soil pressure output for each fatigue load (shown in partial tabular form and graphically):

$q_{r_{north}} =$	$M_{\text{Unique}}_{\text{north}} =$ ·k·ft	$\sigma_{\text{north\_min\_soil}} =$ ·psf	$\sigma_{\text{north\_max\_soiltrap}} =$ ·psf	$\sigma_{\text{north\_max\_soiltri}} =$ ·psf	$\sigma_{\text{north\_max\_soil}} =$ ·psf
0	0	930	930	0	930
1	38	927	933	0	933
2	76	924	935	0	935
3	114	922	938	0	938
4	152	919	941	0	941
5	190	916	944	0	944
6	228	913	947	0	947
7	266	910	949	0	949
8	304	908	952	0	952
9	342	905	955	0	955
10	380	902	958	0	958
11	418	899	960	0	960
12	456	897	963	0	963
13	494	894	966	0	966
14	532	891	969	0	969
15	570	888	972	0	972
...	...	...	...	...	...

### D. Map Unique Matrix Loop Results back to full Minimum Markov or Rain Flow Matrix

$$L_{bmin_{qr}} := \sum_{qr_{north} = 0}^{qt_{north}} \left( \begin{array}{l} \text{Map}L_{b_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = M_{Unique_{north_{qr_{north}}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq M_{Unique_{north_{qr_{north}}}} \end{array} \right)$$

$$\sigma_{minnorth\_max\_soil_{qr}} := \sum_{qr_{north} = 0}^{qt_{north}} \left( \begin{array}{l} \sigma_{north\_max\_soil_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = M_{Unique_{north_{qr_{north}}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq M_{Unique_{north_{qr_{north}}}} \end{array} \right)$$

$$\text{Map}\sigma_{minnorth\_min\_soil_{qr}} := \sum_{qr_{north} = 0}^{qt_{north}} \left( \begin{array}{l} 0 \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"} \\ \text{otherwise} \\ \left| \begin{array}{l} \sigma_{north\_min\_soil_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = M_{Unique_{north_{qr_{north}}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq M_{Unique_{north_{qr_{north}}}} \end{array} \right. \end{array} \right)$$

$$\sigma_{minnorth\_min\_soil_{qr}} := \begin{array}{l} \text{"LIFT"} \text{ if } \text{Map}\sigma_{minnorth\_min\_soil_{qr}} = 0 \\ \text{Map}\sigma_{minnorth\_min\_soil_{qr}} \text{ if } \text{Map}\sigma_{minnorth\_min\_soil_{qr}} \neq 0 \end{array}$$

### E. Map Unique Matrix Loop Results back to full Maximum Markov or Rain Flow Matrix

$$L_{b_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} \text{Map}L_{b_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\sigma_{\text{maxnorth\_max\_soil}_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} \sigma_{\text{north\_max\_soil}_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\text{Map}\sigma_{\text{maxnorth\_min\_soil}_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} 0 \text{ if } \sigma_{\text{north\_min\_soil}_{q_{r_{north}}}} = \text{"LIFT"} \\ \text{otherwise} \\ \sigma_{\text{north\_min\_soil}_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\sigma_{\text{maxnorth\_min\_soil}_{qr}} := \begin{array}{l} \text{"LIFT"} \text{ if } \text{Map}\sigma_{\text{maxnorth\_min\_soil}_{qr}} = 0 \\ \text{Map}\sigma_{\text{maxnorth\_min\_soil}_{qr}} \text{ if } \text{Map}\sigma_{\text{maxnorth\_min\_soil}_{qr}} \neq 0 \end{array}$$



### F. Fatigue Load Bottom Moments and Top Moments at Critical Section

$$\begin{aligned}
 M_{fminbotNorth_{qr}} := & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot (\sigma_{minnorth\_max\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot (\sigma_{minnorth\_max\_soil_{qr}}) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face} \\
 & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}}) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face}
 \end{aligned}$$

$$\begin{aligned}
 M_{fmaxbotNorth_{qr}} := & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_b_{qr}} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{if } \sigma_{maxnorth\_min\_soil_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_b_{qr}} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} \right) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face} \\
 & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}} \right) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face}
 \end{aligned}$$

$$\begin{aligned}
 M_{f_{\text{mintopNorth}}_{\text{qr}}} := & \max \left[ 0 \text{ k}\cdot\text{ft}, \int_a^{D-x_{\text{face\_alt}}} \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{L_{\text{bmin}}_{\text{qr}}} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} \right) \right] \cdot D \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \right. \\
 & + \int_0^a \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{L_{\text{bmin}}_{\text{qr}}} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \int_0^{D-x_{\text{face\_alt}}} \left[ \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \right. \\
 & \quad \left. + \text{DrySoilVolumeFat}(h_b + h_c, y) \gamma_{\text{sdbot}} \dots \right. \\
 & \quad \left. + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \right. \\
 & \quad \left. + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \right. \\
 & \quad \left. + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \right] \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot (D - x_{\text{face\_alt}}) \dots \\
 & + -W_{\text{mean}} \cdot \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c)(h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c - \gamma_{\text{sdbot}})(h_p - h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -M_{\text{minnorth}}_{\text{qr}} \left. \right] \text{ if } \sigma_{\text{min}} \\
 \\
 & \max \left[ 0 \text{ k}\cdot\text{ft}, \int_0^a \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{D} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \sigma_{\text{minnorth\_min\_soil}}_{\text{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \right. \\
 & + \int_a^{D-x_{\text{face\_alt}}} \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{D} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \sigma_{\text{minnorth\_min\_soil}}_{\text{qr}} \right) \right] \cdot D \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \int_0^{D-x_{\text{face\_alt}}} \left[ \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \right. \\
 & \quad \left. + \text{DrySoilVolumeFat}(h_b + h_c, y) \gamma_{\text{sdbot}} \dots \right. \\
 & \quad \left. + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \right. \\
 & \quad \left. + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \right. \\
 & \quad \left. + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \right] \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot (D - x_{\text{face\_alt}}) \dots \\
 & + -W_{\text{mean}} \cdot \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c)(h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c - \gamma_{\text{sdbot}})(h_p - h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots
 \end{aligned}$$

$$M_{fmaxtopNorth_{qr}} := \begin{cases} \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \int_a^{D-x_{face\_alt}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot D \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \int_0^{D-x_{face\_alt}} \left[ \begin{array}{l} \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{array} \right] \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot (D - x_{face\_alt}) \dots \\ + -W_{mean} \cdot \frac{S_{embedding}}{2} \dots \\ + -(\gamma_c)(h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ + -(\gamma_c - \gamma_{sdbot})(h_p - h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ + -M_{maxnorth_{qr}} \end{cases} \quad \text{if } \sigma_{maxnorth\_min\_soil} \\ \\ \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \int_a^{D-x_{face\_alt}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot D \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \int_0^{D-x_{face\_alt}} \left[ \begin{array}{l} \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{array} \right] \cdot (D - x_{face\_alt} - y) \, dy \dots \\ + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot (D - x_{face\_alt}) \dots \\ + -W_{mean} \cdot \frac{S_{embedding}}{2} \dots \\ + -(\gamma_c)(h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ + -(\gamma_c - \gamma_{sdbot})(h_p - h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \end{cases} \quad \text{otherwise}$$

**G. Fatigue Load Shear at Critical Section**

$$\begin{aligned}
 V_{EdminNorth_{qr}} := & \max \left( 0 \text{ kip}, \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \right. \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot D \, dy \dots \\
 & + \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \\
 & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot D \, dy \dots \\
 & + \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots
 \end{aligned}$$

$$\begin{aligned}
 V_{EdmaxNorth_{qr}} := & \left[ + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \right] \\
 & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{if } \sigma_{maxnorth\_min\_soil_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot D \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & ConcreteVolumeFat(y) \cdot \gamma_c \dots \\ & + DrySoilVolumeFat(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + SaturatedSoilVolumeFat(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + VariableSoilWedgeWeightFat[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -BuoyancyWeightFat(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \\
 & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot D \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & ConcreteVolumeFat(y) \cdot \gamma_c \dots \\ & + DrySoilVolumeFat(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + SaturatedSoilVolumeFat(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + VariableSoilWedgeWeightFat[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -BuoyancyWeightFat(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)]
 \end{aligned}$$

## H. Shear and Moment Summary

Results in partial tabular form:

qr =	$V_{EdminNorth} =$	$V_{EdmaxNorth} =$	$M_{fminbotNorth} =$	$M_{fmaxbotNorth} =$	$M_{fmintopNorth} =$	$M_{fmaxtopNorth} =$
	·kip	·kip	·k·ft	·k·ft	·k·ft	·k·ft
0	153	153	1191	1191	0	-1350
1	153	153	1191	1191	0	-1350
2	153	153	1191	1191	0	-1350
3	153	153	1191	1191	0	-1350
4	153	153	1191	1191	0	-1350
5	153	153	1191	1191	0	-1350
6	153	153	1191	1191	0	-1350
7	153	153	1191	1191	0	-1350
8	153	153	1191	1191	0	-1350
9	153	153	1191	1191	0	-1350
10	153	153	1191	1191	0	-1350
11	153	153	1191	1191	0	-1350
12	153	153	1191	1191	0	-1350
13	153	153	1191	1191	0	-1350
14	153	153	1191	1191	0	-1350
15	153	153	1191	1191	0	-1350
16	153	153	1191	1191	0	-1350
17	153	153	1191	1191	0	-1350
18	153	153	1191	1191	0	-1350
19	153	153	1191	1191	0	-1350
20	153	153	1191	1191	0	-1350
21	153	153	1191	1191	0	-1350
22	153	153	1191	1191	0	-1350
23	153	153	1191	1191	0	-1350
24	153	153	1191	1191	0	-1350
25	153	153	1191	1191	0	-1350
26	153	153	1191	1191	0	-1350
...	...	...	...	...	...	...

## I. Transformed Section Analysis at Critical Section

The neutral axis depth in the cracked section is governed by the following cubic equation:

$$\text{CUBIC} := 0$$

Given  $x_{cr} := 11.11 \cdot \text{in}$

$$\text{CUBIC} = \frac{a \cdot x_{cr}^3}{3 \cdot h_c} + \frac{B \cdot x_{cr}^2}{2} - n_{mod} \cdot A_s \cdot (d_{face2} - x_{cr})$$

$$x_{cr} := \text{Find}(x_{cr}) \quad x_{cr} = 12.92 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{CR} := \frac{B \cdot x_{cr}^3}{3} + \frac{2 \cdot \left(\frac{x_{cr} \cdot a}{h_c}\right) \cdot x_{cr}^3}{12} + n_{mod} \cdot A_s \cdot (d_{face2} - x_{cr})^2$$

$$I_{CR} = 60.5 \cdot \text{ft}^4$$

## J. Compute Concrete and Steel Stresses

Elastic beam theory prediction of minimum compressive stress in concrete:

$$\sigma_{cminNorth_{qr}} := \max\left(0 \text{ psi}, \frac{M_{fminbotNorth_{qr}} \cdot x_{cr}}{I_{CR}}\right)$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stminNorth_{qr}} := \max\left[0 \text{ psi}, \frac{n_{mod} \cdot M_{fminbotNorth_{qr}} \cdot (d_{face2} - x_{cr})}{I_{CR}}\right]$$

Elastic beam theory prediction of compressive stress in concrete:

$$\sigma_{cmaxNorth_{qr}} := \frac{M_{fmaxbotNorth_{qr}} \cdot x_{cr}}{I_{CR}}$$

Elastic beam theory prediction of tensile stress in reinforcement:

$$\sigma_{stmaxNorth_{qr}} := \frac{n_{mod} \cdot M_{fmaxbotNorth_{qr}} \cdot (d_{face2} - x_{cr})}{I_{CR}}$$



### K. Compute Twisting Moments and Steel Stresses

Map results from unique matrices back to full matrices:

$$b_{\text{effminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} b_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$x_{\text{start\_minNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} x_{\text{startNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$f_{\text{fminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} f_{\text{fNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$l_{\text{effminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} l_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

Minimum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{\text{twist\_min}_{qr}} := \min(b_{\text{effminNorth}_{qr}}, a - x_{\text{start\_minNorth}_{qr}}) \cdot f_{\text{fminNorth}_{qr}} \cdot \frac{\left( \frac{l_{\text{effminNorth}_{qr}} - C}{2} \right)^2}{2}$$

Map results from unique matrices back to full matrices:

$$b_{\text{effmaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} b_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$x_{\text{start\_maxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} x_{\text{startNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$f_{\text{fimaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} f_{\text{fNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$l_{\text{effimaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} l_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

Maximum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{\text{twist\_max}_{qr}} := \min(b_{\text{effmaxNorth}_{qr}}, a - x_{\text{start\_maxNorth}_{qr}}) \cdot f_{\text{fimaxNorth}_{qr}} \cdot \frac{\left( \frac{l_{\text{effimaxNorth}_{qr}} - C}{2} \right)^2}{2}$$

Area of steel available in sloped foundation section to transfer twisting moments:

$$A_{s\_twist} := \left( \frac{W_m - \frac{B}{2}}{s_{botm}} \right) \cdot (A_{botm}) + \left( \frac{\frac{D}{2} - W_m}{s_{boto}} \right) \cdot (A_{boto}) = 19.3 \cdot \text{in}^2$$

Depth to reinforcement at critical section for flat portion of footing:

$$d_{face\_twist} := h_b + h_c - cc_{bot} - di_{botm} \quad d_{face\_twist} = 57.6 \cdot \text{in}$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

$$\text{QUBIC} := 0$$

Given  $x_{cr2} := 19.54 \cdot \text{in}$

$$\text{QUBIC} = \frac{2x_{cr2}}{3} \cdot \frac{x_{cr2}}{2} \cdot \frac{x_{cr2} \cdot a}{h_c} - n_{mod} \cdot A_{s\_twist} \cdot d_{face\_twist}$$

$$x_{cr2} := \text{Find}(x_{cr2}) \quad x_{cr2} = 17.96 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{CR\_twist} := \frac{\left( x_{cr2} \cdot \frac{a}{h_c} \right) \cdot x_{cr2}^3}{12} + n_{mod} \cdot A_{s\_twist} \cdot (d_{face\_twist} - x_{cr2})^2$$

$$I_{CR\_twist} = 15.3 \cdot \text{ft}^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stmin\_twist\_qr} := \max \left[ 0 \text{psi}, \frac{n_{mod} \cdot M_{twist\_min\_qr} \cdot (d_{face\_twist} - x_{cr2})}{I_{CR\_twist}} \right]$$

Elastic beam theory prediction of tensile stress in reinforcement:

$$\sigma_{stmax\_twist\_qr} := \frac{n_{mod} \cdot M_{twist\_max\_qr} \cdot (d_{face\_twist} - x_{cr2})}{I_{CR\_twist}}$$

### L. Concrete and Flexural Steel Stress Summary

Results in partial tabular form:

qr =	$\sigma_{cminNorth} =$	$\sigma_{cmaxNorth} =$	$\sigma_{stminNorth} =$	$\sigma_{stmaxNorth} =$	$\sigma_{stmin\_twist} =$	$\sigma_{stmax\_twist} =$
0	147 psi	147 psi	4405 psi	4405 psi	14567 psi	14567 psi
1	147	147	4405	4405	14567	14567
2	147	147	4405	4405	14567	14567
3	147	147	4405	4405	14567	14567
4	147	147	4405	4405	14567	14567
5	147	147	4405	4405	14567	14567
6	147	147	4405	4405	14567	14567
7	147	147	4405	4405	14567	14567
8	147	147	4405	4405	14567	14567
9	147	147	4405	4405	14567	14567
10	147	147	4405	4405	14567	14567
11	147	147	4405	4405	14567	14567
12	147	147	4405	4405	14567	14567
13	147	147	4405	4405	14567	14567
14	147	147	4405	4405	14567	14567
15	147	147	4405	4405	14567	14567
16	147	147	4405	4405	14567	14567
17	147	147	4405	4405	14567	14567
18	147	147	4405	4405	14567	14567
19	147	147	4405	4405	14567	14567
20	147	147	4405	4405	14567	14567
21	147	147	4405	4405	14567	14567
22	147	147	4405	4405	14567	14567
23	147	147	4405	4405	14567	14567
24	147	147	4405	4405	14567	14567
25	147	147	4405	4405	14567	14567
26	147	147	4405	4405	14567	14567
27	147	147	4405	4405	14567	14567
28	147	147	4405	4405	14567	14567
...	...	...	...	...	...	...

### M. Check of Fatigue in Concrete due to Compressive Stress

Material Coefficient for Reinforced Concrete:

$$\gamma_{cdnv} := 1.35$$

Characteristic Compressive Cylinder Strength:

$$f_{ck} := f_{ck\_conversion} \cdot f_c \quad f_{ck} = 32.8 \cdot \text{MPa}$$

Normalized Structural Compressive Strength:

$$f_{cn} := f_{ck} \cdot \left( 1 - \frac{f_{ck}}{600 \cdot \text{MPa}} \right) \quad f_{cn} = 31.0 \cdot \text{MPa}$$

Design Concrete Compressive Strength:

$$f_{cd} := \frac{f_{cn}}{\gamma_{cdnv}} \quad f_{cd} = 22.9 \cdot \text{MPa}$$

Ratio between smallest and largest stresses in compression zone:

$$\beta := \text{if}(x_{cr} > 300 \text{mm}, 1, 0) \quad \beta = 1.00$$

Amplification factor to for linear stress distribution in compression zone:

$$\alpha := \max(1.0, 1.3 - 0.3 \cdot \beta) \quad \alpha = 1.00$$

Compressive Strength for Fatigue Check:

$$f_{rd} := \alpha \cdot f_{cd} \quad f_{rd} = 22.9 \cdot \text{MPa}$$

Exposure Factor:

$$C_{1dnv} := 12$$

Fatigue Strength Parameter:

$$C_{5c} := 1.0$$

Cycles Allowed:

$$n_{callowNorth_{qr}} := 10 \cdot C_{1dnv} \cdot \left( \frac{1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}} \right)$$

$$X_{dnvNorth_{qr}} := \frac{C_{1dnv}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2dnvNorth_{nr}} := \max \left[ 1 + 0.2 \cdot (\log(n_{callowNorth_{nr}}) - X_{dnvNorth_{nr}}), 1.0 \right]$$

$$n_{callowNorth_{qr}} := \text{if} \left[ \log(n_{callowNorth_{qr}}) > X_{dnvNorth_{qr}}, 10 \cdot C_{1dnv} \cdot C_{2dnvNorth_{qr}} \cdot \left( \frac{1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}} \right), n_{callowNorth_{qr}} \right]$$

Damage:

$$\text{Damage}_{North_{qr}} := \frac{N_{fat_{qr}}}{n_{callowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108):

$$\text{Damage}_{totalNorth} := \sum \text{Damage}_{North}$$

$$\text{Damage}_{totalNorth} = 0.00$$

$$\text{if}(\text{Damage}_{totalNorth} \leq 0.33, \text{"OK"}, \text{"NG"}) = \text{"OK"}$$

## N. Check of Fatigue in Concrete due to Shear

Concrete Area Resisting Shear:  $A_{\text{face}} = 137 \text{ ft}^2$

Characteristic Tensile Strength:  $f_{\text{tk}} := 0.48 \cdot (f_{\text{cck}} \cdot \text{MPa})^{0.5}$   $f_{\text{tk}} = 2.75 \cdot \text{MPa}$

Normalized Tensile Strength:  $f_{\text{tn}} := f_{\text{tk}} \cdot \left[ 1 - \left( \frac{f_{\text{tk}}}{25 \cdot \text{MPa}} \right)^{0.6} \right]$   $f_{\text{tn}} = 2.02 \cdot \text{MPa}$

Design Tensile Strength:  $f_{\text{td}} := \frac{f_{\text{tn}}}{\gamma_{\text{cdnv}}}$   $f_{\text{td}} = 1.49 \cdot \text{MPa}$

Design Constants:  $k_A := 100 \cdot \text{MPa}$

$d_1 := 1000 \cdot \text{mm}$

Anchored Reinforcement on Tensile Side:  $A_s = 63 \cdot \text{in}^2$

Design Factor:  $k_v := \min \left( \max \left( 1.5 - \frac{d_{\text{face}2}}{d_1}, 1.0 \right), 1.4 \right)$   $k_v = 1.00$

Design Shear Strength:  $V_{\text{cd}} := \min \left[ 0.3 \cdot \left( f_{\text{td}} + \frac{k_A \cdot A_s}{\gamma_{\text{cdnv}} \cdot A_{\text{face}}} \right) \cdot A_{\text{face}} \cdot k_v, 0.6 \cdot f_{\text{td}} \cdot A_{\text{face}} \cdot k_v \right]$

$V_{\text{cd}} = 1488 \cdot \text{k}$

Design Shear Strength Stated in Terms of Stress:  $v_{\text{cd}} := \min \left[ 0.3 \cdot \left( f_{\text{td}} + \frac{k_A \cdot A_s}{\gamma_{\text{cdnv}} \cdot A_{\text{face}}} \right) \cdot k_v, 0.6 \cdot f_{\text{td}} \cdot k_v \right]$   $v_{\text{cd}} = 75 \text{ psi}$

Cycles Allowed:  $n_{\text{vcallowNorth}_{\text{qr}}} := 10$

$$C_{1\text{dnv}} \cdot \frac{\left( 1 - \frac{V_{\text{EdmaxNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} \right)}{\left( 1 - \frac{V_{\text{EdminNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} \right)}$$

$$X_{\text{dnvNorth}_{\text{qr}}} := \frac{C_{1\text{dnv}}}{1 - \frac{V_{\text{EdminNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} + 0.1 \cdot C_{1\text{dnv}}}$$

$$C_{2\text{dnvNorth}_{\text{qr}}} := \max \left[ 1 + 0.2 \cdot \left( \log \left( n_{\text{vcallowNorth}_{\text{qr}}} \right) - X_{\text{dnvNorth}_{\text{qr}}} \right), 1.0 \right]$$

$$n_{vallowNorth_{qr}} := \text{if} \left[ \log \left( n_{vallowNorth_{qr}} \right) > X_{dvnNorth_{qr}}, 10, n_{vallowNorth_{qr}} \right]$$

$$C_{1dvn} \cdot C_{2dvnNorth_{qr}} \cdot \frac{\left( \frac{V_{EdmaxNorth_{qr}}}{1 - \frac{V_{EdmaxNorth_{qr}}}{C_{5c} \cdot V_{cd}}} \right)}{\left( \frac{V_{EdminNorth_{qr}}}{1 - \frac{V_{EdminNorth_{qr}}}{C_{5c} \cdot V_{cd}}} \right)}$$

Damage:

$$Damage_{vNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{vallowNorth_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$Damage_{vtotalNorth} := \sum Damage_{vNorth}$$

$$Damage_{vtotalNorth} = 0.00$$

$$\text{if} (Damage_{vtotalNorth} \leq 0.33, "OK", "NG") = "OK"$$

### O. Check of Fatigue in Grout Bearing Stress

Material Coefficient for Plain Grout:  $\gamma_{gdnv} := 1.35$

Exposure Factor:  $C_{1gdnv} := 12$

Fatigue Strength Parameter:  $C_{5g} := 0.8$

Characteristic Compressive Grout Strength:  $f_{grtk} := f_{cck\_conversion} \cdot f_{c28}$   $f_{grtk} = 52.4 \cdot \text{MPa}$

Normalized Structural Compressive Grout Strength:  $f_{gn} := f_{grtk} \cdot \left(1 - \frac{f_{grtk}}{600 \cdot \text{MPa}}\right)$   $f_{gn} = 47.8 \cdot \text{MPa}$

Design Concrete Compressive Grout Strength:  $f_{gd} := \frac{f_{gn}}{\gamma_{gdnv}}$   $f_{gd} = 35.4 \cdot \text{MPa}$

Anchor Bolt Pretension Load:  $T_{pre} = 72 \cdot \text{kip}$

Flange Outside Diameter:  $OD = 4556 \cdot \text{mm}$

Flange Inside Diameter:  $ID = 4000 \cdot \text{mm}$

Number of Bolts:  $N = 140$

PVC outside diameter:  $d_{SDR} = 1.660 \cdot \text{in}$

Bearing area at top of grout:  $A_{grt} := \frac{\pi}{4} \cdot (OD^2 - ID^2) - N \cdot \frac{\pi \cdot d_{SDR}^2}{4}$   $A_{grt} = 5488 \cdot \text{in}^2$

Section modulus at top of grout:  $S_{grt} := \frac{\pi}{32 \cdot OD} (OD^4 - ID^4) + \left[ \frac{2}{OD} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \right]$   $S_{grt} = 217939 \cdot \text{in}^3$

Minimum grout fatigue stress due to wind:  $b_{fat\_grt\_minnorth\_qr} := \frac{(M_{fatminnorth\_qr})}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$

Maximum grout fatigue stress due to wind:  $b_{fat\_grt\_maxnorth\_qr} := \frac{(M_{fatmaxnorth\_qr})}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$



Cycles Allowed:

$$n_{brg\_grt\_allowNorth_{qr}} := 10 \cdot C_{1gdnv} \cdot \frac{\left( \frac{b_{fat\_grt\_maxnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}{\left( \frac{b_{fat\_grt\_minnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}$$

$$X_{brg\_grt\_dnvNorth_{qr}} := \frac{C_{1gdnv}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}} + 0.1 \cdot C_{1gdnv}}$$

$$C_{2brg\_grt\_dnvNorth_{qr}} := \max \left[ 1 + 0.2 \cdot \left( \log \left( n_{brg\_grt\_allowNorth_{qr}} \right) - X_{brg\_grt\_dnvNorth_{qr}} \right), 1.0 \right]$$

$$n_{brg\_grt\_allowNorth_{qr}} := \text{if} \left[ \log \left( n_{brg\_grt\_allowNorth_{qr}} \right) > X_{brg\_grt\_dnvNorth_{qr}}, 10 \cdot C_{1gdnv} \cdot C_{2brg\_grt\_dnvNorth_{qr}} \cdot \frac{\left( \frac{b_{fat\_grt\_maxnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}{\left( \frac{b_{fat\_grt\_minnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}, n_{brg\_grt\_allowNorth_{qr}} \right]$$

Damage:

$$\text{Damage}_{brg\_grt\_North_{qr}} := \frac{N_{fat_{qr}}}{n_{brg\_grt\_allowNorth_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$\text{Damage}_{brg\_grt\_totalNorth} := \sum \text{Damage}_{brg\_grt\_North}$$

$$\text{Damage}_{brg\_grt\_totalNorth} = 0.00$$

$$\text{if} \left( \text{Damage}_{brg\_grt\_totalNorth} \leq 1.0, \text{"OK"} , \text{"NG"} \right) = \text{"OK"}$$

## P. Check of Fatigue in Pedestal Bearing Stress

Characteristic Compressive  
Cylinder Strength:

$$f_{cckp} := f_{cck\_conversion} \cdot f_{cp} \quad f_{cckp} = 32.8 \cdot \text{MPa}$$

Normalized Structural  
Compressive Strength:

$$f_{cnp} := f_{cckp} \cdot \left( 1 - \frac{f_{cckp}}{600 \cdot \text{MPa}} \right) \quad f_{cnp} = 31.0 \cdot \text{MPa}$$

Design Concrete  
Compressive Strength:

$$f_{cdp} := \frac{f_{cnp}}{\gamma_{cdnv}} \quad f_{cdp} = 22.9 \cdot \text{MPa}$$

Grout thickness:

$$t_g = 0.00 \cdot \text{in}$$

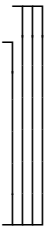
Bearing area at bottom of grout:

$$A_1 := \frac{\pi}{4} \cdot \left[ (OD + t_g)^2 - (ID - t_g)^2 \right] - N \cdot \frac{\pi \cdot d_{SDR}^2}{4} \quad A_1 = 5488 \cdot \text{in}^2$$

Section modulus at bottom of grout:

$$S_1 := \frac{\pi}{32 \cdot (OD + t_g)} \left[ (OD + t_g)^4 - (ID - t_g)^4 \right] \dots$$

$$+ \left[ \frac{2}{(OD + t_g)} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \dots \right. \right. \right]$$

$$\left. \left. + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \quad S_1 = 217939 \cdot \text{in}^3$$


Minimum fatigue stress due to wind:

$$b_{fatminnorth_{qr}} := \frac{(M_{fatminnorth_{qr}})}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$$

Maximum fatigue stress due to wind:

$$b_{fatmaxnorth_{qr}} := \frac{(M_{fatmaxnorth_{qr}})}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$$

Bearing strip (radial) at bottom of grout:

$$A_{strip\_1} := \frac{(OD + t_g) - (ID - t_g)}{2} \quad A_{strip\_1} = 10.9 \cdot \text{in}$$

Angle of bearing within concrete:

$$\alpha_{DNV} := 0 \text{ deg}$$

$$\alpha_{DNV} = 0.0 \text{ deg}$$

Critical bearing angle within concrete that defines the pedestal bottom edge:

$$\alpha_{critical} := \min \left[ \text{atan} \left[ \frac{C - (OD + t_g)}{2} \right], \text{atan} \left( \frac{1}{2} \right) \right]$$

$$\alpha_{critical} = 23.7 \text{ deg}$$

Bearing strip (radial) within concrete at base of pedestal:

$$A_{strip\_2} := \min(A_{strip\_1} + 2 \cdot h_p \cdot \tan(\min(\alpha_{DNV}, \alpha_{critical})), 4 \cdot A_{strip\_1}, A_{strip\_1} + h_p)$$

$$A_{strip\_2} = 10.9 \text{ in}$$

Design Bearing Capacity Stated in Terms of Stress:

$$F_{cd} := f_{cdp} \cdot \min \left[ \left( \frac{A_{strip\_2}}{A_{strip\_1}} \right)^{\frac{1}{3}}, 1.3 \right]$$

$$F_{cd} = 3326 \text{ psi}$$

Cycles Allowed:

$$C_{1dnv} := \frac{\left( 1 - \frac{b_{fatmaxnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}{\left( 1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}$$

$$n_{brgallowNorth\_qr} := 10$$

$$X_{brgdnvNorth\_qr} := \frac{C_{1dnv}}{1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2brgdnvNorth\_qr} := \max \left[ 1 + 0.2 \cdot (\log(n_{brgallowNorth\_qr}) - X_{brgdnvNorth\_qr}), 1.0 \right]$$

$$n_{brgallowNorth\_qr} := \text{if} \left[ \log(n_{brgallowNorth\_qr}) > X_{brgdnvNorth\_qr}, 10, C_{1dnv} \cdot C_{2brgdnvNorth\_qr} \cdot \frac{\left( 1 - \frac{b_{fatmaxnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}{\left( 1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}, n_{brgallowNorth\_qr} \right]$$

Damage:

$$\text{Damage}_{brgNorth\_qr} := \frac{N_{fat\_qr}}{n_{brgallowNorth\_qr}}$$

Accumulated Damage (Section 6, M108):

$$\text{Damage}_{brgtotalNorth} := \sum \text{Damage}_{brgNorth}$$

$$\text{Damage}_{brgtotalNorth} = 0.07$$

$$\text{if}(\text{Damage}_{brgtotalNorth} \leq 0.33, \text{"OK"}, \text{"NG"}) = \text{"OK"}$$

## R. Check of Fatigue in Bottom Steel due to Tensile Stress - Primary Direction

Design factors:  $C_3 = 19.60$

$$C_4 = 6.00$$

Tensile stress range in steel:  $\Delta\sigma_{stNorth_{qr}} := \max\left[1 \cdot \text{psi}, \left(\sigma_{stmaxNorth_{qr}} - \sigma_{stminNorth_{qr}}\right)\right]$

Characteristic strength of reinforcement:  $f_{sk} := f_{yB}$

Material coefficient for reinforcement:  $\gamma_s = 1.00$

Check maximum tensile stress in steel:  $\text{CheckStress}_{Northmax2} := \text{if}\left(\max(\sigma_{stmaxNorth}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"} , \text{"Okay"}\right) = \text{"Okay"}$

Cycles Allowed:  $n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{stNorth_{qr}}}{\text{MPa}}\right)\right)}$

$$n_{sallowNorth_{qr}} := \text{if}\left(n_{sallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:  $\text{Damage}_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$

Accumulated Damage (Section 6, M108):  $\text{Damage}_{stotalNorth} := \sum \text{Damage}_{sNorth}$

$$\text{Damage}_{stotalNorth} = 0.00$$

$$\text{if}\left(\text{Damage}_{stotalNorth} \leq 0.33, \text{"OK"} , \text{"NG"}\right) = \text{"OK"}$$

### S. Check of Fatigue in Bottom Steel due to Tensile Stress - Normal Direction

Design factors:  $C_3 = 19.60$

$C_4 = 6.00$

Tensile stress range in steel:  $\Delta\sigma_{stNorth\_normal\_qr} := \max\left[1 \cdot \text{psi}, \left(\sigma_{stmax\_twist\_qr} - \sigma_{stmin\_twist\_qr}\right)\right]$

Characteristic strength of reinforcement:  $f_{sk} := f_{yB}$

Material coefficient for reinforcement:  $\gamma_s = 1.00$

Check maximum tensile stress in steel:  $\text{CheckStress}_{Northmax3} := \text{if}\left(\max(\sigma_{stmax\_twist}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"} , \text{"Okay"}\right) = \text{"Okay"}$

Cycles Allowed:  $n_{sallowNorth\_qr} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{stNorth\_normal\_qr}}{\text{MPa}}\right)\right)}$

$n_{sallowNorth\_qr} := \text{if}\left(n_{sallowNorth\_qr} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth\_qr}\right)$

Damage:  $\text{Damage}_{sNorth\_qr} := \frac{N_{fat\_qr}}{n_{sallowNorth\_qr}}$

Accumulated Damage (Section 6, M108):  $\text{Damage}_{stotalNorth\_normal} := \sum \text{Damage}_{sNorth}$

$\text{Damage}_{stotalNorth\_normal} = 0.00$

$\text{if}\left(\text{Damage}_{stotalNorth\_normal} \leq 0.33, \text{"OK"} , \text{"NG"}\right) = \text{"OK"}$

## T. Check of Fatigue in Top Steel due to Tensile Stress

Depth to reinforcement at critical section for flat portion of footing:

$$d_{\text{face2}} := h_b + h_c - cc_{\text{top}} - 1.5d_{\text{topm}} - 1 \text{ in} \quad d_{\text{face2}} = 57.9 \cdot \text{in}$$

Depth to reinforcement at critical section for sloped portion of footing:

$$d_{\text{face3}} := \frac{d_{\text{edge}} + d_{\text{face2}}}{2} \quad d_{\text{face3}} = 51.23 \cdot \text{in}$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

Given  $x_{\text{cr}} := 11.49 \cdot \text{in}$

$$\text{CUBIC} = \frac{w_f \cdot x_{\text{cr}}^2}{2} - n_{\text{mod}} \cdot \left( \frac{B}{s_{\text{topm}}} \cdot A_{\text{topm}} \right) \cdot (d_{\text{face2}} - x_{\text{cr}}) \dots$$

$$+ -n_{\text{mod}} \cdot \left[ \frac{(w_f - B)}{s_{\text{topm}}} \cdot A_{\text{topm}} \right] \cdot (d_{\text{face3}} - x_{\text{cr}})$$

$$x_{\text{cr}} := \text{Find}(x_{\text{cr}}) \quad x_{\text{cr}} = 6.95 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{\text{CR}} := \frac{w_f \cdot x_{\text{cr}}^3}{3} + n_{\text{mod}} \cdot \left( \frac{B}{s_{\text{topm}}} \cdot A_{\text{topm}} \right) \cdot (d_{\text{face2}} - x_{\text{cr}})^2 \dots$$

$$+ n_{\text{mod}} \cdot \left[ \frac{(w_f - B)}{s_{\text{topm}}} \cdot A_{\text{topm}} \right] \cdot (d_{\text{face3}} - x_{\text{cr}})^2$$

$$I_{\text{CR}} = 22.8 \cdot \text{ft}^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{\text{stmintopNorth}_{\text{qr}}} := \frac{n_{\text{mod}} \cdot M_{\text{fmintopNorth}_{\text{qr}}} \cdot (d_{\text{face2}} - x_{\text{cr}})}{I_{\text{CR}}}$$

Elastic beam theory prediction of maximum tensile stress in reinforcement:

$$\sigma_{\text{stmaxtopNorth}_{\text{qr}}} := \frac{n_{\text{mod}} \cdot M_{\text{fmaxtopNorth}_{\text{qr}}} \cdot (d_{\text{face2}} - x_{\text{cr}})}{I_{\text{CR}}}$$

Design factors:

$$C_3 = 19.60$$

$$C_4 = 6.00$$

Tensile stress range in steel:

$$\Delta\sigma_{\text{sttopNorth}_{qr}} := \max\left[1 \cdot \text{psi}, \left(\sigma_{\text{stmaxtopNorth}_{qr}} - \sigma_{\text{stmintopNorth}_{qr}}\right)\right]$$

Characteristic strength of reinforcement:

$$f_{sk} := f_{yT}$$

Material coefficient for reinforcement:

$$\gamma_s = 1.00$$

Check maximum tensile stress in steel:

$$\text{CheckStress}_{\text{Northmax4}} := \text{if}\left(\max(\sigma_{\text{stmaxtopNorth}}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}$$

Cycles Allowed:

$$n_{\text{sallowNorth}_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{\text{sttopNorth}_{qr}}}{\text{MPa}}\right)\right)}$$

$$n_{\text{sallowNorth}_{qr}} := \text{if}\left(n_{\text{sallowNorth}_{qr}} > 2 \cdot 10^8, 10^{307}, n_{\text{sallowNorth}_{qr}}\right)$$

Damage:

$$\text{Damage}_{\text{sNorth}_{qr}} := \frac{N_{\text{fat}_{qr}}}{n_{\text{sallowNorth}_{qr}}}$$

Accumulated Damage  
(Section 6, M107):

$$\text{Damage}_{\text{stoptotalNorth}} := \sum \text{Damage}_{\text{sNorth}}$$

$$\text{Damage}_{\text{stoptotalNorth}} = 0.48$$

$$\text{if}\left(\text{Damage}_{\text{stoptotalNorth}} \leq 0.33, \text{"OK"}, \text{"NG"}\right) = \text{"NG"}$$

### U. Check Fatigue Pullout Strength of Concrete (concrete only)

Minimum fatigue force due to wind:

$$P_{fat\_cone\_min\_qr} := \max \left[ 0 \text{ psi}, \left[ \frac{4 \cdot (M_{minnorth\_qr})}{\frac{N}{2} \cdot D_{avg}} - \frac{W_{mean} + W_p}{\frac{N}{2}} \right] \right]$$

Maximum fatigue force due to wind:

$$P_{fat\_cone\_max\_qr} := \left[ \frac{4 \cdot (M_{maxnorth\_qr})}{\frac{N}{2} \cdot D_{avg}} - \frac{W_{mean} + W_p}{\frac{N}{2}} \right]$$

Area of Cone

$$A_{cone} = 991.59 \cdot \text{in}^2$$

Maximum stress  
in concrete shear cone:

$$\sigma_{cone\_max\_qr} := \frac{P_{fat\_cone\_max\_qr}}{A_{cone}}$$

$$\max(\sigma_{cone\_max}) = 65.35 \text{ psi}$$

Minimum stress in  
concrete shear cone:

$$\sigma_{cone\_min\_qr} := \frac{P_{fat\_cone\_min\_qr}}{A_{cone}}$$

$$\max(\sigma_{cone\_min}) = 62.23 \text{ psi}$$

Characteristic Tensile Strength:

$$f_{tk} := 0.48 \cdot (f_{cck} \cdot \text{MPa})^{0.5}$$

$$f_{tk} = 2.75 \cdot \text{MPa}$$

Normalized Tensile Strength:

$$f_{tn} := f_{tk} \cdot \left[ 1 - \left( \frac{f_{tk}}{25 \cdot \text{MPa}} \right)^{0.6} \right] = 292.51 \text{ psi}$$

$$f_{tn} = 2.02 \cdot \text{MPa}$$

Design Tensile Strength:

$$f_{td} := \frac{f_{tn}}{\gamma_{cdnv}}$$

$$f_{td} = 1.49 \cdot \text{MPa}$$

Design Constants:

$$k_A := 100 \cdot \text{MPa}$$

$$d_1 := 1000 \cdot \text{mm}$$

Calibration Factor:

$$U := 0.79$$

$$U \cdot f_{td} = 171 \text{ psi}$$

Additional Reduction factor:

$$\gamma_{sf} := 1.35$$

Design Shear Strength  
Stated in Terms of Stress:

$$v_{cd\_cone} := \frac{U \cdot f_{td} \cdot k_v}{\gamma_{sf}}$$

$$v_{cd\_cone} = 127 \text{ psi}$$

Cycles Allowed:

$$n_{cone\_allow\_qr} := 10 \cdot C_{1dnt} \cdot \frac{\left( 1 - \frac{\sigma_{cone\_max\_qr}}{C_{5c} \cdot v_{cd\_cone}} \right)}{\left( 1 - \frac{\sigma_{cone\_min\_qr}}{C_{5c} \cdot v_{cd\_cone}} \right)}$$



$$X_{dnv_{qr}} := \frac{C_{1dnv}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2dnv_{nr}} := \max\left[1 + 0.2 \cdot (\log(n_{cone\_allow_{nr}}) - X_{dnv_{nr}}), 1.0\right]$$

$$n_{cone\_allow_{qr}} := \text{if} \left[ \log(n_{cone\_allow_{qr}}) > X_{dnv_{qr}}, 10, \left[ C_{1dnv} \cdot C_{2dnv_{qr}} \cdot \frac{\left( \frac{\sigma_{cone\_max_{qr}}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}}} \right)}{\left( \frac{\sigma_{cone\_min_{qr}}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}}} \right)}, n_{cone\_allow_{qr}} \right] \right]$$

Damage:

$$Damage_{cone_{qr}} := \frac{N_{fat_{qr}}}{n_{cone\_allow_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$Damage_{cone\_total} := \sum Damage_{cone}$$

$$Damage_{cone\_total} = 0.00$$

$$\text{if}(Damage_{cone\_total} < 1.0, "OK", "NG") = "OK"$$



### III. Foundation, Tower and Design Information

#### A. Unit Definitions and Foundation Dimensions

$k \equiv 1000 \cdot \text{lbf}$       meter  $\equiv$  m

Foundation width:	$D := 50 \cdot \text{ft}$	Pedestal diameter:	$C := 17 \cdot \text{ft}$	Average extension of pedestal above ground surface:	$h_{pe} := 6 \text{ in} + \left( \frac{0.25 \text{ in}}{\text{ft}} \right) \cdot \left( \frac{D - C}{4} \right)$
Height of base:	$h_b := 28 \cdot \text{in}$	Height of center (above base):	$h_c := 34 \cdot \text{in}$		$h_{pe} = 8.06 \cdot \text{in}$
Height of soil (from foundation bottom):	$h_s := 7.5 \text{ ft} - h_{pe}$ $h_s = 81.94 \cdot \text{in}$	Height of pedestal:	$h_p := h_s - h_b - h_c + h_{pe}$ $h_p = 28.00 \cdot \text{in}$		



Height of embedment ring above bottom of footing:

$h_c := 10 \cdot \text{in}$

Minimum depth of groundwater below grade:

$d_{GWT} := 3 \cdot \text{ft}$

Minimum depth of groundwater below grade (fatigue):

$d_{GWTf} := 3 \cdot \text{ft}$

Top width:

$B := \frac{D}{1 + \sqrt{2}}$

$B = 20.71 \text{ ft}$

Side width:

$a := \frac{D - B}{2}$

$a = 14.64 \text{ ft}$

Edge slope:

$\xi_1 := \text{atan} \left( \frac{h_c}{a} \right)$

$\xi_1 = 10.95 \cdot \text{deg}$

Corner slope:

$\xi_2 := \text{atan} \left( \frac{2h_c}{B} \right)$

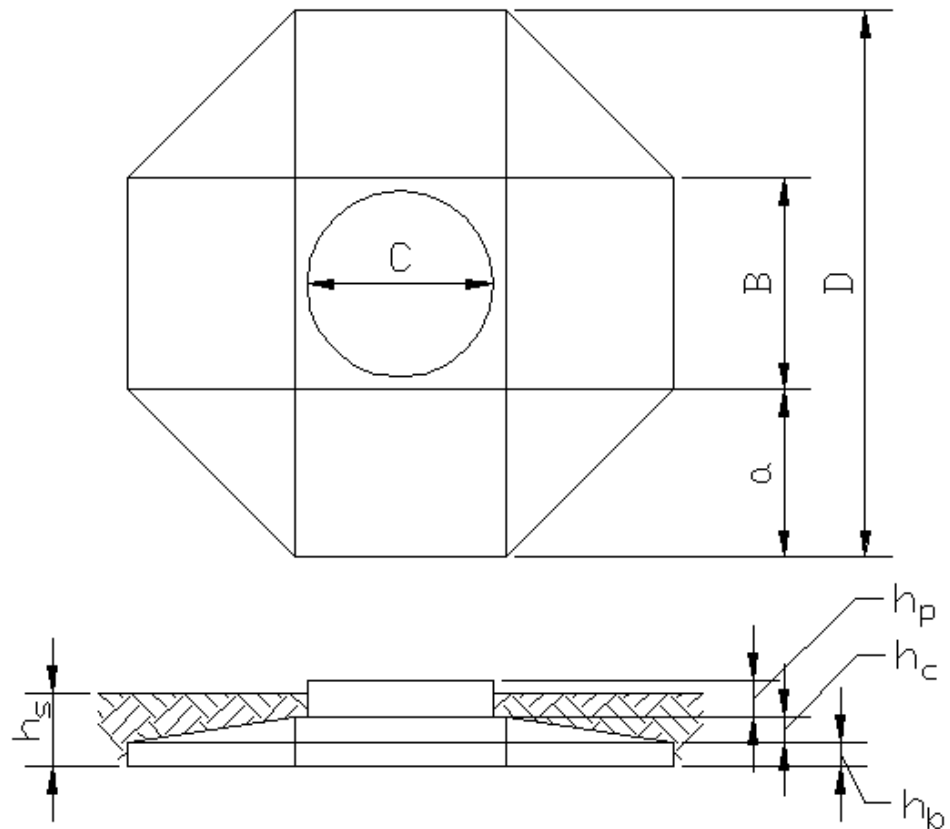
$\xi_2 = 15.30 \cdot \text{deg}$

Bottom cover:

$cc_{bot} := 3 \cdot \text{in}$

Top cover:

$cc_{top} := 2 \cdot \text{in}$



GroundwaterLocation :=  $\left\{ \begin{array}{l} \text{if } d_{GWT} < h_s \\ \text{"Within the Base"} \text{ if } d_{GWT} \geq h_s - h_b \\ \text{if } d_{GWT} < h_s - h_b \\ \text{"Within the Center"} \text{ if } d_{GWT} \geq h_s - h_b - h_c \\ \text{"Within the Pedestal"} \text{ otherwise} \\ \text{"Below the Foundation"} \text{ otherwise} \end{array} \right.$

GroundwaterLocation = "Within the Center"

### B. Tower Bottom Flange Dimensions

Flange outside diameter:	$OD := 4556 \cdot \text{mm}$	$OD = 14.95 \text{ ft}$
Flange inside diameter:	$ID := 4000 \cdot \text{mm}$	$ID = 13.12 \text{ ft}$
Total number of bolts (2 circles):	$N := 2 \cdot 70$	
Inner bolt circle diameter:	$D_i := 4130 \cdot \text{mm}$	$D_i = 13.55 \text{ ft}$
Outer bolt circle diameter:	$D_o := 4426 \cdot \text{mm}$	$D_o = 14.52 \text{ ft}$
Average tower diameter:	$d_{\text{tower}} := 4300 \cdot \text{mm} - 26 \text{mm}$	$d_{\text{tower}} = 14.02 \text{ ft}$
Thickness of tower flange:	$t_{\text{flange}} := 75 \cdot \text{mm}$	$t_{\text{flange}} = 2.95 \cdot \text{in}$
Average bolt circle diameter:	$D_{\text{avg}} := \frac{D_i + D_o}{2}$	$D_{\text{avg}} = 168.43 \cdot \text{in}$
Width of flange:	$w_{\text{flange}} := \frac{OD - ID}{2}$	$w_{\text{flange}} = 10.94 \cdot \text{in}$

### C. Stability Safety Factors

Minimum factor of safety:	$FS_{\text{min}} := 1.5$	(Reference 1)
Minimum factor of safety:	$FS_{\text{min}2} := 1.0$	(Reference 1)

### D. Stiffness Requirements

Required dynamic rotational stiffness:	$K_{\psi \text{req}} := \frac{5.0 \cdot 10^7 \cdot \text{kN} \cdot \text{m}}{\text{rad}}$	(Reference 3)
Required dynamic translational stiffness:	$K_{x \text{req}} := \frac{1.0 \cdot 10^6 \cdot \text{kN}}{\text{m}}$	(Reference 3)

### E. ACI Reinforcing Information

(Reference 1a)

Bar nominal size, diameter (in), area (in <sup>2</sup> ), and weight (lbf/ft):	ACI_bar_table :=	<table> <tr> <td>"not used"</td> <td>0.0001</td> <td>0</td> <td>0</td> </tr> <tr> <td>3</td> <td>0.375</td> <td>0.11</td> <td>0.376</td> </tr> <tr> <td>4</td> <td>0.500</td> <td>0.20</td> <td>0.668</td> </tr> <tr> <td>5</td> <td>0.625</td> <td>0.31</td> <td>1.043</td> </tr> <tr> <td>6</td> <td>0.750</td> <td>0.44</td> <td>1.502</td> </tr> <tr> <td>7</td> <td>0.875</td> <td>0.60</td> <td>2.044</td> </tr> <tr> <td>8</td> <td>1.000</td> <td>0.79</td> <td>2.670</td> </tr> <tr> <td>9</td> <td>1.128</td> <td>1.00</td> <td>3.400</td> </tr> <tr> <td>10</td> <td>1.270</td> <td>1.27</td> <td>4.303</td> </tr> <tr> <td>11</td> <td>1.410</td> <td>1.56</td> <td>5.313</td> </tr> <tr> <td>14</td> <td>1.693</td> <td>2.25</td> <td>7.650</td> </tr> <tr> <td>18</td> <td>2.257</td> <td>4.00</td> <td>13.600</td> </tr> </table>	"not used"	0.0001	0	0	3	0.375	0.11	0.376	4	0.500	0.20	0.668	5	0.625	0.31	1.043	6	0.750	0.44	1.502	7	0.875	0.60	2.044	8	1.000	0.79	2.670	9	1.128	1.00	3.400	10	1.270	1.27	4.303	11	1.410	1.56	5.313	14	1.693	2.25	7.650	18	2.257	4.00	13.600
	"not used"	0.0001	0	0																																														
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	7	0.875	0.60	2.044																																														
	8	1.000	0.79	2.670																																														
	9	1.128	1.00	3.400																																														
	10	1.270	1.27	4.303																																														
	11	1.410	1.56	5.313																																														
14	1.693	2.25	7.650																																															
18	2.257	4.00	13.600																																															

## F. Material Properties

Friction factor:	$\mu_f := 0.4$	
Concrete strength:	$f_c := 5000 \cdot \text{psi}$	
Steel yield strength (bottom mat):	$f_{yB} := 75000 \text{psi}$	
Steel yield strength (top mat):	$f_{yT} := 75000 \text{psi}$	
Steel verts yield strength:	$f_{yV} := 60000 \cdot \text{psi}$	
Steel modulus of elasticity:	$E_s := 29000 \text{ksi}$	
Density of concrete:	$\gamma_c := 150 \text{pcf}$	
Density of water:	$\gamma_w := 62.4 \text{pcf}$	
Dry density of soil:	$\gamma_d := 105 \text{pcf}$	
Design density of soil above groundwater table:	$\gamma_{sdbot} := \gamma_d \cdot (1 + 15.5\%)$	$\gamma_{sdbot} = 121 \cdot \text{pcf}$
	$\gamma_{sdtop} := \gamma_{sdbot}$	
Specific gravity of soil solids:	$G_s := 2.70$	
Design density of soil below groundwater table:	$\gamma_{ssbot} := \left(1 - \frac{1}{G_s}\right) \cdot \gamma_d + \gamma_w$	$\gamma_{ssbot} = 129 \cdot \text{pcf}$
	$\gamma_{sstop} := \min(120 \text{pcf}, \gamma_{ssbot})$	
Soil wedge angle from vertical:	$\theta := \text{atan}\left(\frac{1}{2}\right)$	$\theta = 26.6 \cdot \text{deg}$
Soil wedge angle from vertical (fatigue):	$\theta_{fat} := \text{atan}\left(\frac{0}{2}\right)$	$\theta_{fat} = 0.00 \cdot \text{deg}$
Concrete modulus of elasticity:	$E_c := 57000 \text{psi} \cdot \sqrt{\frac{f_c}{\text{psi}}}$	$E_c = 4031 \cdot \text{ksi}$
Modulus reduction factor:	$\psi := 0.8$	(Reference 7)
Conversion factor for concrete and grout strength fatigue calculations:	$f_{cck\_conversion} := (1 - 5\%)$	$f_{cck\_conversion} = 0.95$
Modular ratio:	$n_{mod} := \frac{E_s}{\psi \cdot E_c}$	$n_{mod} = 9.0$

## G. Extreme Loading Conditions

(Reference 3)

### Misalignment Loading

Base moment:  $M_{align} := 965 \text{ kN}\cdot\text{m}$

$$M_{align} = 712 \cdot \text{ft}\cdot\text{k}$$

Misalignment  
angle (relative to  
 $\Delta := 45 \text{ deg}$

### Normal Extreme

Normal extreme load factor:

$$\alpha_e := 1.35$$

Base moment:

$$M_e := \frac{41406.2}{\alpha_e} \text{ kN}\cdot\text{m}$$

$$M_e = 22622 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_e := \frac{546}{\alpha_e} \text{ kN}$$

$$H_e = 91 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{te} := \frac{2725.2}{\alpha_e} \text{ kN}$$

$$W_{te} = 454 \cdot \text{k}$$

### Abnormal Extreme

Abnormal extreme load factor:

$$\alpha_a := 1.10$$

Base moment:

$$M_a := \frac{37785.1 \text{ kN}\cdot\text{m}}{\alpha_a}$$

$$M_a = 25335 \cdot \text{k}\cdot\text{ft}$$

Base shear:

$$H_a := \frac{526.8 \text{ kN}}{\alpha_a}$$

$$H_a = 108 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{ta} := \frac{2163.3 \text{ kN}}{\alpha_a}$$

$$W_{ta} = 442 \cdot \text{k}$$

## H. Normal Loading Conditions - DLC 1.0

Base moment:

$$M_N := 19257.1 \cdot \text{kN}\cdot\text{m}$$

$$M_N = 14203 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_N := 285.7 \cdot \text{kN}$$

$$H_N = 64 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_N := 2029.4 \cdot \text{kN}$$

$$W_N = 456 \cdot \text{k}$$

## I. Normal Loading Conditions - DLC 1.1

Base moment:

$$M_{1.1} := 19257.1 \cdot \text{kN}\cdot\text{m}$$

$$M_{1.1} = 14203 \cdot \text{ft}\cdot\text{k}$$

Base shear:

$$H_{1.1} := 285.7 \cdot \text{kN}$$

$$H_{1.1} = 64 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{1.1} := 2029.4 \cdot \text{kN}$$

$$W_{1.1} = 456 \cdot \text{k}$$

## J. Earthquake Loading Conditions

(Reference 1)

Seismic Design Criteria:

Site<sub>Class</sub> := "D"

- Building Occupancy Category: II (Non-Essential Power Facility, Non Hazardous)

- Seismic Design Category: A (Determination of Seismic Design Category)

0.2 Second spectral response:  $S_S := 0.069$

1.0 Second spectral response:  $S_1 := 0.033$

**Table 11.4-1 Site Coefficient,  $F_a$**

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at Short Period				
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.2	1.2	1.1	1	1
D	1.6	1.4	1.2	1.1	1
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

$F_a = 1.60$  4 #N/A #N/A

**Table 11.4-2 Site Coefficient,  $F_v$**

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1	1	1	1	1
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

$F_v = 2.40$  #N/A

Importance factor:  $I := 1.00$

Adjusted 0.2s response:  $S_{MS} := F_a \cdot S_S$   $S_{MS} = 0.11$

Adjusted 1.0s response:  $S_{M1} := F_v \cdot S_1$   $S_{M1} = 0.08$

Design response:  $S_{DS} := \frac{2}{3} \cdot S_{MS}$   $S_{DS} = 0.07$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$   $S_{D1} = 0.05$

Vertical seismic load effect:  $E_v := \text{if}(S_{DS} \leq 0.125, 0, 0.2 \cdot S_{DS})$   $E_v = 0.00$

Seismic Design Requirements for Nonbuilding/Building Structures:

Response modification coefficient:  $R := 1.5$

Long period transition period:  $T_L := 12$

Period coefficients:  $C_t := 0.02$

$x_{EQ} := 0.75$

Height of structure:  $h_n := 80 \cdot \text{m}$

Approximate fundamental period of structure:  $T_{a1} := C_t \cdot \left(\frac{h_n}{\text{ft}}\right)^{x_{EQ}} \cdot \text{s}$   $T_{a1} = 1.3 \text{ s}$

Value of  $C_u$  is calculated based on ASCE 7-05 Table 12.8-1

$C_u = 1.70$

Table 12.8-1 COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD	
Design Spectral Response Acceleration Parameter at 1 s, SD1	Coefficient $C_u$
$\geq 0.4$	1.4
0.3	1.4
0.2	1.5
0.15	1.6
$\leq 0.1$	1.7

Maximum approximate period:  $T_{a2} := C_u \cdot T_{a1}$   $T_{a2} = 2.22 \text{ s}$

Design period:  $T := \min(T_{a1}, T_{a2})$   $T = 1.3 \text{ s}$

Seismic response coefficients:  $C_{s1} := \frac{S_{DS}}{\frac{R}{I}}$   $C_{s1} = 0.049$

$$C_{s2} := \text{if} \left[ \frac{T}{\text{sec}} \leq T_L, \frac{S_{D1}}{\left(\frac{T}{\text{sec}}\right) \cdot \left(\frac{R}{I}\right)}, \frac{S_{D1} \cdot T_L}{\left(\frac{T}{\text{sec}}\right)^2 \cdot \left(\frac{R}{I}\right)} \right] = 0.027$$

Non-Building structure:  $C_{s3} := 0.03$

$$C_{s4} := \text{if} \left( S_1 > 0.6, \frac{0.8 \cdot S_1}{\frac{R}{I}}, 0 \right) \quad C_{s4} = 0.00$$

$C_s := \max(\min(C_{s1}, C_{s2}), C_{s3}, C_{s4})$   $C_s = 0.03$

Minimum base shear:  $V := C_s \cdot W_{1.1}$   $V = 14 \cdot \text{kip}$

Structure Weights and Centers of Gravity:

(Reference 3)

Total weight of structure:

$$W_{1,1} = 456 \cdot \text{kip}$$

Weight of Component

Approximate Center of Gravity

Tower bottom section::

$$W_1 := 121500 \text{ lbf}$$

$$h_1 := \frac{21925 \text{ mm}}{2}$$

Tower lower mid section:

$$W_2 := 112240 \text{ lbf}$$

$$h_2 := \frac{25912 \text{ mm}}{2} + 21925 \text{ mm}$$

Tower upper mid section:

$$W_3 := 104090 \text{ lbf}$$

$$h_3 := \frac{29463 \text{ mm}}{2} + 25912 \text{ mm} + 21925 \text{ mm}$$

Tower top section:

$$W_4 := 0 \cdot \text{lbf}$$

$$h_4 := 0 \cdot \text{mm}$$

Wind turbine nacelle and rotor:

$$h_5 := 80 \cdot \text{m}$$

Total tower weight:

$$W_{\text{tower}} := W_1 + W_2 + W_3 + W_4$$

$$W_{\text{tower}} = 338 \cdot \text{kip}$$

Wind turbine nacelle and rotor:

$$W_5 := W_{1,1} - W_{\text{tower}}$$

$$W_5 = 118 \cdot \text{kip}$$

Summary:

$$W = \begin{pmatrix} 0 \\ 122 \\ 112 \\ 104 \\ 0 \\ 118 \end{pmatrix} \cdot \text{kip}$$

$$h = \begin{pmatrix} 0.0 \\ 11.0 \\ 34.9 \\ 62.6 \\ 0.0 \\ 80.0 \end{pmatrix} \cdot \text{m}$$



Determine Design Base Shear:

Exponent related to period of structure:

$$k_0 := \text{if} \left[ \frac{T}{\text{sec}} \leq 0.5, 1, \text{if} \left[ \frac{T}{\text{sec}} \geq 2.5, 2, 2 - \left( \frac{2.5 - \frac{T}{\text{sec}}}{2} \right) \right] \right]$$

$$k_0 = 1.4$$

Number of tower components:

$$\text{ntc} := 1 \dots 5$$

Vertical Distribution Factor:

$$C_{v_{\text{ntc}}} := \frac{W_{\text{ntc}} \cdot \left( \frac{h_{\text{ntc}}}{\text{ft}} \right)^{k_0}}{\sum_{i=1}^5 \left[ W_i \cdot \left( \frac{h_i}{\text{ft}} \right)^{k_0} \right]}$$

Lateral Seismic Force at Each Level:

$$F_{\text{ntc}} := C_{v_{\text{ntc}}} \cdot V$$

$$C_{v_{\text{ntc}}} =$$

0.032
0.149
0.314
0.000
0.504

Total lateral seismic force:

$$F_{\text{ntc}} =$$

0.4	·kip
2.0	
4.3	
0.0	
6.9	

$$H_{\text{EQ}} := \sum_{i=1}^5 (F_i)$$

$$H_{\text{EQ}} = 14 \cdot \text{kip}$$

Earthquake overturning moment:

$$M_{\text{EQ}} := \sum_{i=1}^5 (F_i \cdot h_i)$$

$$M_{\text{EQ}} = 2945 \cdot \text{ft} \cdot \text{kip}$$

Base moment:

$$M_{\text{OE}} := \sqrt{M_{1.1}^2 + M_{\text{EQ}}^2}$$

$$M_{\text{OE}} = 14505 \cdot \text{ft} \cdot \text{k}$$

Base shear:

$$H_{\text{OE}} := \sqrt{H_{1.1}^2 + H_{\text{EQ}}^2}$$

$$H_{\text{OE}} = 66 \cdot \text{k}$$

Tower & turbine dead weight:

$$W_{\text{OE}} := W_{1.1}$$

$$W_{\text{OE}} = 456 \cdot \text{k}$$

### K. Fatigue Loading Conditions

(Reference 3)

Mean shear:

$$H_{\text{mean}} := \sqrt{213.7^2 + 3.7^2} \text{ kN} = 213.73 \cdot \text{kN}$$

$$H_{\text{mean}} = 214 \cdot \text{kN}$$

Mean overturning moment:

$$M_{\text{mean}} := \sqrt{1270.2^2 + 15989.9^2} \text{ kN}\cdot\text{m} = 16040.27 \cdot \text{kN}\cdot\text{m}$$

$$M_{\text{mean}} = 16040 \cdot \text{kN}\cdot\text{m}$$

Turbine & tower mean weight:

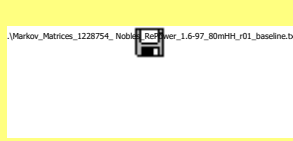
$$W_{\text{mean}} := 2011.2 \text{ kN}$$

$$W_{\text{mean}} = 452 \cdot \text{kip}$$

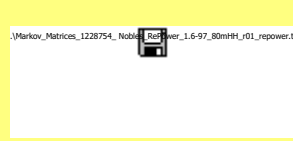
Fatigue Loading (Markov Matrix):

input raw Markov Matrix data - width/range, mean, cycles

baseline :=



repower :=



$$\text{rows}(\text{repower}) = 47247$$

$$\text{rows}(\text{baseline}) + \text{rows}(\text{repower}) = 87242$$

$$\text{markov\_baseline} := \text{augment}(\text{baseline}^{\langle 0 \rangle}, \text{baseline}^{\langle 1 \rangle}, \text{baseline}^{\langle 2 \rangle})$$

$$\text{markov\_repower} := \text{augment}(\text{repower}^{\langle 0 \rangle}, \text{repower}^{\langle 1 \rangle}, \text{repower}^{\langle 2 \rangle})$$

Years of operation on baseline turbine:

$$\text{Year}_{\text{baseline}} := 11$$

Years of fatigue assumed baseline turbine:

$$\text{Year}_{\text{fat\_baseline}} := 11$$

$$N_{\text{fat\_baseline}} := \left( \frac{\text{Year}_{\text{baseline}}}{\text{Year}_{\text{fat\_baseline}}} \right) \cdot \text{baseline}^{\langle 2 \rangle}$$

$$N_{\text{fat\_baseline}_{50}} = 424000$$

Years of operation on repower turbine:

$$\text{Year}_{\text{repower}} := 20$$

Years of fatigue assumed repower turbine:

$$\text{Year}_{\text{fat\_repower}} := 20$$

$$N_{\text{fat\_repower}} := \left( \frac{\text{Year}_{\text{repower}}}{\text{Year}_{\text{fat\_repower}}} \right) \cdot \text{repower}^{\langle 2 \rangle}$$

$$N_{\text{fat\_repower}_{105}} = 638000$$

Minimum Bins

$$\text{MinBin\_baseline} := \text{baseline}^{\langle 1 \rangle} - \frac{\text{baseline}^{\langle 0 \rangle}}{2}$$

$$\text{MinBin\_repower} := \text{repower}^{\langle 1 \rangle} - \frac{\text{repower}^{\langle 0 \rangle}}{2}$$

$$N_{\text{fat}} := \text{stack}(N_{\text{fat\_baseline}}, N_{\text{fat\_repower}}) \quad \text{rows}(N_{\text{fat}}) = 87242$$

$$N_{\text{fat}_{509}} = 417000$$

Maximum Bins

$$\text{MaxBin\_baseline} := \text{baseline}^{\langle 1 \rangle} + \frac{\text{baseline}^{\langle 0 \rangle}}{2}$$

$$\text{MaxBin\_repower} := \text{repower}^{\langle 1 \rangle} + \frac{\text{repower}^{\langle 0 \rangle}}{2}$$

$$\text{MinBin\_total} := \text{stack}(\text{MinBin\_baseline}, \text{MinBin\_repower})$$

$$\text{rows}(\text{MinBin\_total}) = 87242$$

$$\text{MaxBin\_total} := \text{stack}(\text{MaxBin\_baseline}, \text{MaxBin\_repower})$$

$$\text{rows}(\text{MaxBin\_total}) = 87242$$

Bin counters:

$$qt := \text{length}(N_{fat}) - 1 = 87241$$

$$qr := 0, 1 .. qt$$

North Bins

$$\text{MinBin}_{qr} := \text{if}(\text{MinBin}_{total_{qr}} < 0, 0, \text{MinBin}_{total_{qr}})$$

$$\text{MaxBin}_{qr} := \text{if}(\text{MaxBin}_{total_{qr}} < 0, 0, \text{MaxBin}_{total_{qr}})$$

Minimum Moment (for Miner's Rule):

$$M_{fatminnorth_{qr}} := \text{MinBin}_{qr} \cdot \text{kN} \cdot \text{m}$$

$$H_{minnorth_{qr}} := \max\left(\frac{M_{fatminnorth_{qr}}}{M_{mean}} \cdot H_{mean}, 0.01 \text{ kN}\right)$$

$$M_{minnorth_{qr}} := M_{fatminnorth_{qr}} + H_{minnorth_{qr}} \cdot (h_p + h_c + h_b)$$

Maximum Moments:

$$M_{maxfatigue} := \max(M_{maxnorth})$$

$$M_{minfatigue} := \max(M_{minnorth})$$

Fatigue moments are plotted below:

$$\text{Doubleqr} := 0, 1 .. 2 \cdot qt + 1$$

South Bins

$$\text{MinBin}_{qr} := \text{if}(\text{MaxBin}_{total_{qr}} > 0, 0, -1 \cdot \text{MaxBin}_{total_{qr}})$$

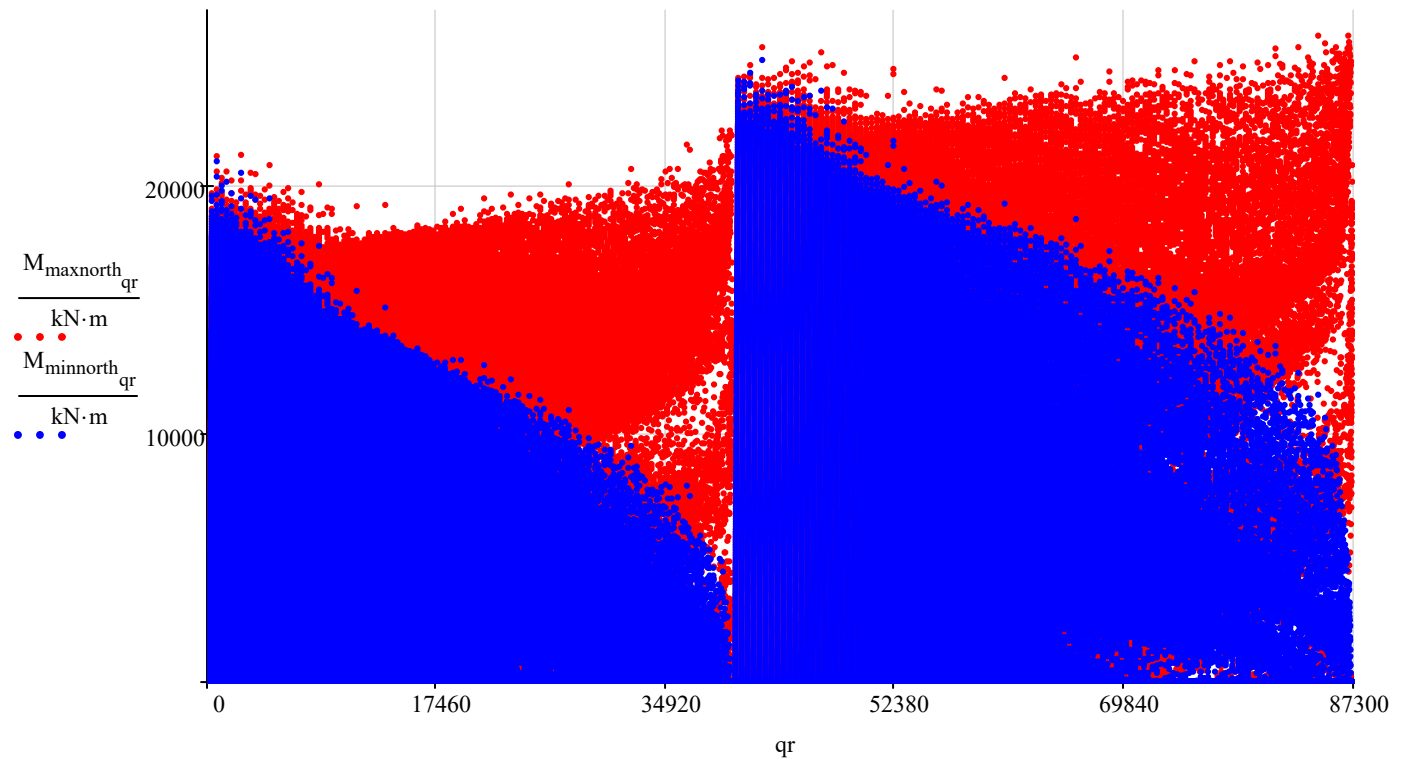
$$\text{MaxBin}_{qr} := \text{if}(\text{MinBin}_{total_{qr}} > 0, 0, -1 \cdot \text{MinBin}_{total_{qr}})$$

Maximum Moment (for Miner's Rule):

$$M_{fatmaxnorth_{qr}} := \text{MaxBin}_{qr} \cdot \text{kN} \cdot \text{m}$$

$$H_{maxnorth_{qr}} := \max\left(\frac{M_{fatmaxnorth_{qr}}}{M_{mean}} \cdot H_{mean}, 0.01 \text{ kN}\right)$$

$$M_{maxnorth_{qr}} := M_{fatmaxnorth_{qr}} + H_{maxnorth_{qr}} \cdot (h_p + h_c + h_b)$$

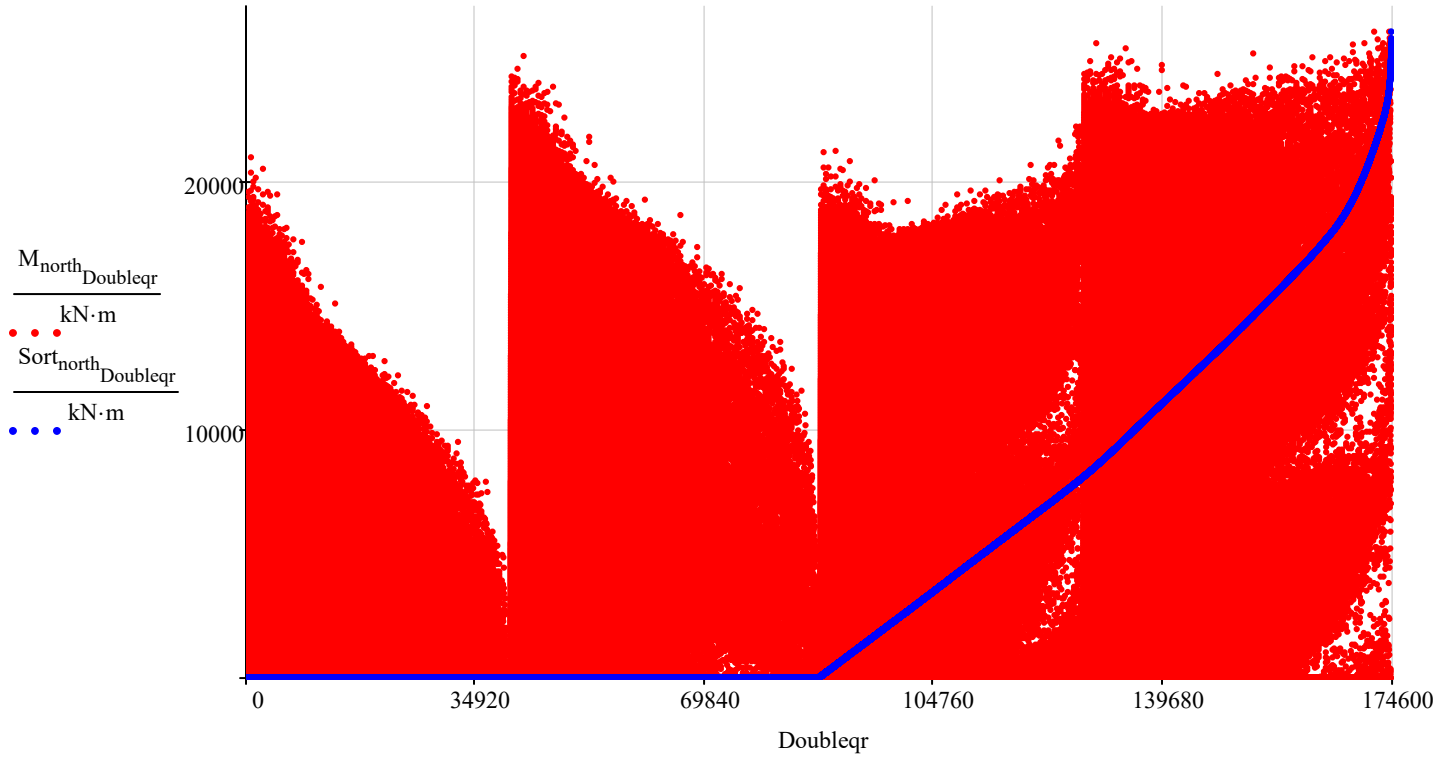


**L. Stack Min and Max Fatigue Matrices and Sort**

Stack min and max  
 fatigue matrices  
 and sort in  
 ascending order:

$$M_{north\_Doubleqr} := \begin{cases} M_{minnorth\_Doubleqr} & \text{if } Doubleqr \leq qt \\ M_{maxnorth\_Doubleqr-qt-1} & \text{if } Doubleqr > qt \end{cases}$$

$$Sort_{north} := \text{sort}(M_{north})$$



### M. Develop Unique Fatigue Matrix and Parse Zero Values

Develop matrix containing only the unique fatigue values:

$$\text{Unique}_{\text{north}_{\text{Doubleqr}}} := \begin{cases} \text{Sort}_{\text{north}_0} & \text{if Doubleqr} = 0 \\ \text{Sort}_{\text{north}_{\text{Doubleqr}}} & \text{if Doubleqr} \neq 0 \wedge \text{Sort}_{\text{north}_{\text{Doubleqr}}} \neq \text{Sort}_{\text{north}_{\text{Doubleqr}-1}} \\ 0 & \text{if Doubleqr} \neq 0 \wedge \text{Sort}_{\text{north}_{\text{Doubleqr}}} = \text{Sort}_{\text{north}_{\text{Doubleqr}-1}} \end{cases}$$

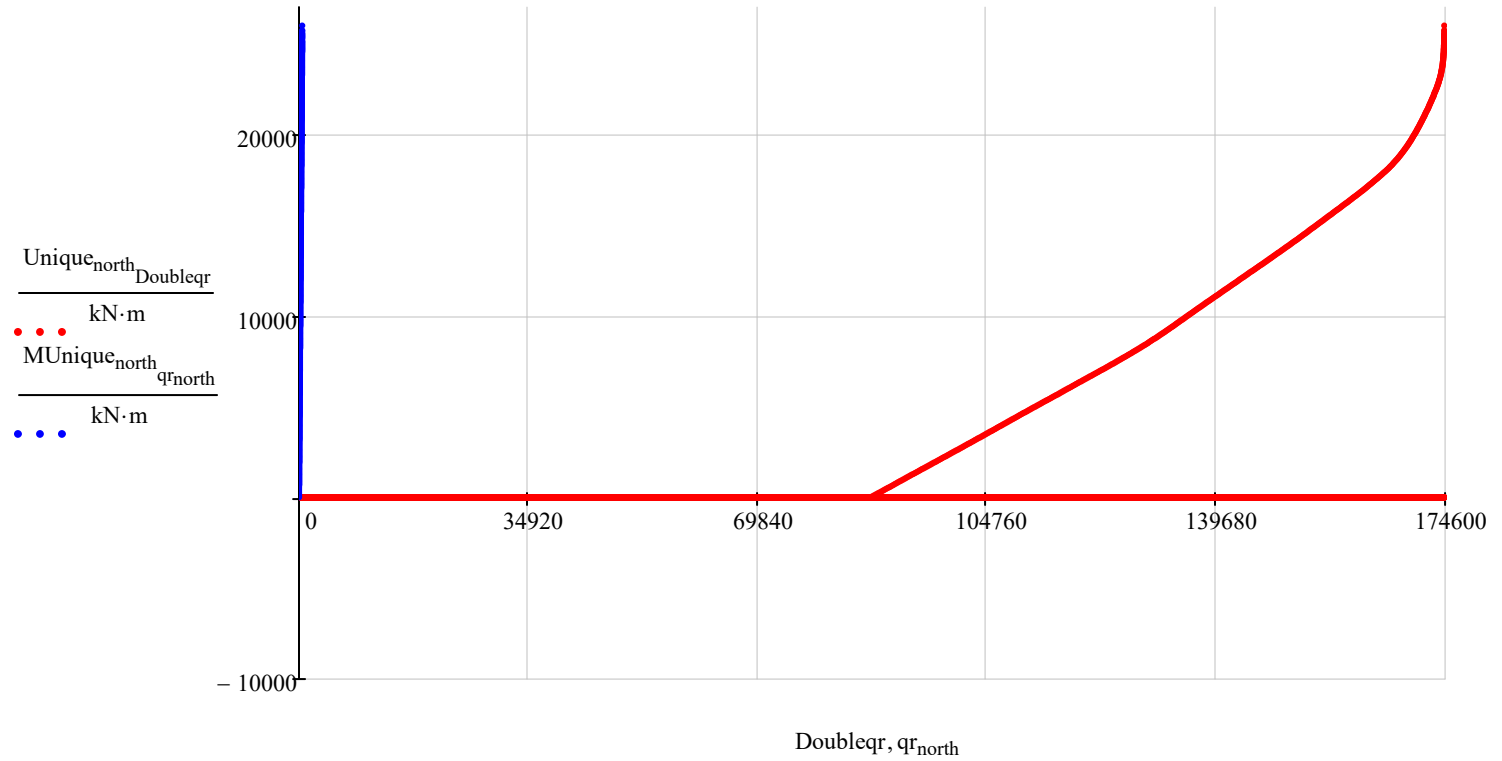
Remove all "zero" entries leaving only "non-zero" entries in the MUnique matrix:

$$\text{MUnique}_{\text{north}} := \begin{cases} \text{trim}(\text{Unique}_{\text{north}}, \text{match}(0, \text{Unique}_{\text{north}})) & \text{if } \min(\text{Unique}_{\text{north}}) = 0 \\ \text{Unique}_{\text{north}} & \text{if } \min(\text{Unique}_{\text{north}}) > 0 \end{cases}$$

Determine the quantity of "non-zero" entries in the MUnique matrix to use as counters:

$$\text{qt}_{\text{north}} := \text{length}(\text{MUnique}_{\text{north}}) - 1 = 498$$

$$\text{qr}_{\text{north}} := 0, 1 \dots \text{qt}_{\text{north}}$$



## IV. Stability Analysis and Bearing Length Evaluation

### A. Foundation Volume and Weight Calculations

Foundation plan area:

$$A := D^2 - 2 \cdot \left( \frac{D - B}{2} \right)^2 \quad A = 2071 \cdot \text{ft}^2$$

Volume of pedestal:

$$v_p := \frac{\pi \cdot C^2}{4} \cdot h_p \quad v_p = 20 \cdot \text{yd}^3$$

Weight of pedestal:

$$W_p := v_p \cdot \gamma_c \quad W_p = 79 \cdot \text{k}$$

Volume of footing:

$$v_f := A \cdot h_b + B^2 \cdot h_c + 4 \cdot \left( \frac{1}{2} \cdot \frac{1}{3} \cdot h_c \cdot a^2 + \frac{1}{2} \cdot B \cdot h_c \cdot a \right) \quad v_f = 303 \cdot \text{yd}^3$$

Weight of footing:

$$W_f := v_f \cdot \gamma_c \quad W_f = 1226 \cdot \text{k}$$

Total volume of concrete:

$$v_c := v_f + v_p \quad v_c = 322 \cdot \text{yd}^3$$

Total volume of soil:

$$v_s := A \cdot h_s - v_f - \frac{\pi \cdot C^2}{4} \cdot (h_s - h_b - h_c) \dots + \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot (h_s - h_b)^2 \quad v_s = 238 \cdot \text{yd}^3$$

Total volume of soil (fatigue)

$$v_{\text{sfat}} := A \cdot h_s - v_f - \frac{\pi \cdot C^2}{4} \cdot (h_s - h_b - h_c) \dots + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot (h_s - h_b)^2 \quad v_{\text{sfat}} = 207 \cdot \text{yd}^3$$

Equivalent buoyant force:

$$F_b := \gamma_w \cdot \begin{cases} \text{if } d_{\text{GWT}} < h_s \\ \left| \begin{array}{l} A \cdot (h_s - d_{\text{GWT}}) \text{ if } d_{\text{GWT}} \geq h_s - h_b \\ A \cdot (h_s - d_{\text{GWT}}) + \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot (h_s - h_b - d_{\text{GWT}})^2 \text{ otherwise} \end{array} \right. \\ 0 \text{ otherwise} \end{cases} \quad F_b = 501 \cdot \text{kip}$$

Equivalent buoyant force (fatigue):

$$F_{\text{bfat}} := \gamma_w \cdot \begin{cases} \text{if } d_{\text{GWTF}} < h_s \\ \left| \begin{array}{l} A \cdot (h_s - d_{\text{GWTF}}) \text{ if } d_{\text{GWTF}} \geq h_s - h_b \\ A \cdot (h_s - d_{\text{GWTF}}) + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \text{ otherwise} \end{array} \right. \\ 0 \text{ otherwise} \end{cases} \quad F_{\text{bfat}} = 495 \cdot \text{kip}$$

Total weight of soil:

$$W_s := \begin{cases} \text{if } d_{GWT} < h_s - h_b \\ \left[ \begin{aligned} &\gamma_{sdbot} \cdot d_{GWT} \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) + \gamma_{ssbot} \cdot (h_s - h_b - h_c - d_{GWT}) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots \\ &+ \gamma_{ssbot} \cdot [A \cdot (h_b + h_c) - v_f] \dots \\ &+ \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ssbot} (h_s - h_b - d_{GWT})^2 + \gamma_{sdbot} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] \\ &\gamma_{sdbot} \cdot (h_s - h_b - h_c) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots \\ &+ \gamma_{sdbot} \cdot \int_{h_s - h_b - h_c}^{d_{GWT}} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \dots \\ &+ \gamma_{ssbot} \cdot \left[ \int_{d_{GWT}}^{h_s - h_b} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{pe}) \right]^2 \right] dy \dots \right. \\ &\left. + \frac{8 \cdot B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ssbot} (h_s - h_b - d_{GWT})^2 + \gamma_{sdbot} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] \right] \\ &v_s \cdot \gamma_{sdbot} \text{ otherwise} \end{aligned} \right] \end{cases} \quad \begin{matrix} \text{if } d_{GWT} < h_s - h_b - h_c \\ \\ \\ \\ \text{otherwise} \end{matrix}$$

$$W_s = 786 \cdot k$$

Total weight of soil (fatigue):

$$W_{\text{sfat}} := \begin{cases} \text{if } d_{\text{GWTF}} < h_s - h_b \\ \gamma_{\text{sdbot}} \cdot d_{\text{GWTF}} \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) + \gamma_{\text{ssbot}} \cdot (h_s - h_b - h_c - d_{\text{GWTF}}) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots & \text{if } d_{\text{GWTF}} < h_s - h_b - h_c \\ + \gamma_{\text{ssbot}} \cdot \left[ A \cdot (h_b + h_c) - v_f \right] \dots \\ + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot \left[ \gamma_{\text{ssbot}} (h_s - h_b - d_{\text{GWTF}})^2 + \gamma_{\text{sdbot}} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] \\ \\ \gamma_{\text{sdbot}} \cdot (h_s - h_b - h_c) \cdot \left( A - \frac{\pi \cdot C^2}{4} \right) \dots & \text{otherwise} \\ + \gamma_{\text{sdbot}} \cdot \int_{h_s - h_b - h_c}^{d_{\text{GWTF}}} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{\text{pe}}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{\text{pe}}) \right]^2 \right] dy \dots \\ + \gamma_{\text{ssbot}} \cdot \left[ \int_{d_{\text{GWTF}}}^{h_s - h_b} 4 \cdot B \cdot \left[ a - \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{\text{pe}}) \right] + 2 \cdot \left[ a^2 - \left[ \left( \frac{a}{h_c} \right) \cdot (y - h_p + h_{\text{pe}}) \right]^2 \right] dy \dots \right. \\ \left. + \frac{8 \cdot B \cdot \tan(\theta_{\text{fat}})}{2} \cdot \left[ \gamma_{\text{ssbot}} (h_s - h_b - d_{\text{GWTF}})^2 + \gamma_{\text{sdbot}} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] \right] \end{cases}$$

$v_{\text{sfat}} \cdot \gamma_{\text{sdbot}}$  otherwise

$$W_{\text{sfat}} = 683744 \cdot \text{lbf}$$

Total dead weight seismic load:

$$W_{\text{EQ}} := W_p + W_f + W_{\text{OE}} + W_s - F_b$$

$$W_{\text{EQ}} = 2047 \cdot \text{k}$$

Total dead weight fatigue load:

$$W_{\text{fat}} := W_p + W_f + W_{\text{mean}} + W_{\text{sfat}} - F_{\text{bfat}}$$

$$W_{\text{fat}} = 1946 \cdot \text{k}$$



## B. Stability Calculations - Extreme Loading

Determine controlling extreme load case:

$$\text{if}(M_e > M_a, \text{"Normal Controls"}, \text{"Abnormal Controls"}) = \text{"Abnormal Controls"}$$

Extreme Tower Base Moment:

$$M := \sqrt{(\text{if}(M_e > M_a, M_e, M_a) + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} = 25844 \cdot \text{k} \cdot \text{ft}$$

Extreme Tower Base Shear:

$$H := \text{if}(M_e > M_a, H_e, H_a) \quad H = 108 \cdot \text{k}$$

Extreme Tower Weight:

$$W_t := \text{if}(M_e > M_a, W_{te}, W_{ta}) \quad W_t = 442 \cdot \text{k}$$

Total dead weight wind load:

$$W_W := W_p + W_f + W_t + W_s - F_b \quad W_W = 2033 \cdot \text{k}$$

Overturning wind moment:

$$M_{oW} := M + (h_b + h_c + h_p) \cdot H \quad M_{oW} = 26651 \cdot \text{k} \cdot \text{ft}$$

Wind load friction resistance at base:

$$H_{frW} := \mu_f \cdot (W_W) \quad H_{frW} = 813 \cdot \text{k}$$

Factor of safety against sliding:

$$FS_{sW} := \frac{H_{frW}}{H} \quad FS_{sW} = 7.55$$

Seismic load friction resistance at base:

$$H_{frEQ} := \mu_f \cdot (W_{EQ}) \cdot (1 - E_v) \quad H_{frEQ} = 819 \cdot \text{k}$$

Factor of safety against sliding:

$$FS_{sEQ} := \frac{H_{frEQ}}{H_{OE}} \quad FS_{sEQ} = 12.47$$

Determine controlling load case:

$$FS_s := \min(FS_{sW}, FS_{sEQ}) \quad FS_s = 7.55$$

$$\text{if}(FS_s \geq FS_{\min}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$$

Resisting moment:

$$M_{rW} := W_W \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right) \quad M_{rW} = 50818 \cdot \text{ft} \cdot \text{k}$$

Factor of safety against overturning:

$$FS_{oW} := \frac{M_{rW}}{M_{oW}} \quad FS_{oW} = 1.91$$

Overturning seismic moment:

$$M_{oEQ} := \sqrt{(M_{OE} + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_{OE} = 15510 \cdot \text{k} \cdot \text{ft}$$

Resisting moment:

$$M_{rEQ} := (1 - E_v) \cdot W_{EQ} \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right) \quad M_{rEQ} = 51170 \cdot \text{ft} \cdot \text{k}$$

Factor of safety against overturning:

$$FS_{oEQ} := \frac{M_{rEQ}}{M_{oEQ}} \quad FS_{oEQ} = 3.30$$

Minimum factor of safety:

$$FS_{\min} = 1.50$$

Determine controlling load case:

$$FS_o := \min(FS_{oW}, FS_{oEQ}) \quad FS_o = 1.91$$

$$\text{if}(FS_o \geq FS_{\min}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$$

Resisting moment (reduced):  $M_{rW\_red} := 0.6W_W \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$   $M_{rW\_red} = 30491 \cdot \text{ft}\cdot\text{k}$

Factor of safety against overturning (alternate):  $FS_{oW\_alt} := \frac{M_{rW\_red}}{M_{oW}}$   $FS_{oW\_alt} = 1.14$

Overturning seismic moment (reduced):  $M_{oEQ\_alt} := 0.7 \cdot M_{oEQ}$   $M_{oEQ\_alt} = 10857 \cdot \text{k}\cdot\text{ft}$

Resisting moment (reduced):  $M_{rEQ\_red} := 0.6(1 - E_v) \cdot W_{EQ} \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$   $M_{rEQ\_red} = 30702 \cdot \text{ft}\cdot\text{k}$

Factor of safety against overturning (alternate):  $FS_{oEQ\_alt} := \frac{M_{rEQ\_red}}{M_{oEQ\_alt}}$   $FS_{oEQ\_alt} = 2.83$

Minimum factor of safety:  $FS_{min2} = 1.00$

Determine controlling load case:  $FS_{o2} := \min(FS_{oW\_alt}, FS_{oEQ\_alt})$   $FS_{o2} = 1.14$

$\text{if}(FS_{o2} \geq FS_{min2}, \text{"OK"}, \text{"No Good"}) = \text{"OK"}$

### C. Soil Pressure Calculations - Extreme Loading

Side length of square inscribed inside of foundation octagon:  $S_f := \sqrt{2a^2 + 2 \cdot a \cdot B + B^2}$   $S_f = 38.3 \text{ ft}$

"Major" width of octagon along widest section:  $H_f := \sqrt{B^2 + D^2}$   $H_f = 54.1 \text{ ft}$

Distance from widest section of octagon to edge of square:  $W_e := \frac{H_f - S_f}{2}$   $W_e = 7.93 \text{ ft}$

Moment of inertia for octagon about any axis through centroid:  $I_{fdn} := \frac{8 \cdot B^4}{192} \cdot \cot\left(\frac{2 \cdot \pi}{16}\right) \left(3 \cot\left(\frac{2 \cdot \pi}{16}\right)^2 + 1\right)$   $I_{fdn} = 342112 \cdot \text{ft}^4$

Section modulus of foundation for normal orientation:  $S_{normal} := \frac{2I_{fdn}}{D}$   $S_{normal} = 13684 \cdot \text{ft}^3$

Section modulus of foundation for orientation rotated by 22.5 degrees:  $S_{rotated} := \frac{2 \cdot I_{fdn}}{H_f}$   $S_{rotated} = 12643 \cdot \text{ft}^3$

$\text{if}(S_{normal} > S_{rotated}, \text{"Rotated Controls"}, \text{"Normal Controls"}) = \text{"Rotated Controls"}$

### 1) Assumed Soil Pressure Case 1

Assume triangular distribution with length of bearing ( $L_b$ ) greater than  $W_e$  but less than or equal to half the major octagon width ( $H_f/2$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2033 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 28354 \cdot ft \cdot k$$

Guess:  $L := \frac{2W_e + H_f}{4} \quad f_{max} := 5000 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^L f_{max} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_4 = 32.3 \text{ ft} \quad f_4 = 3764 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil Bearing Length:  $L_{b4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, L_4, 0 \cdot ft \right) \quad L_{b4} = 0.0 \cdot ft$

Maximum Soil Bearing Pressure:  $f_{max4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, f_4, 0 \cdot psf \right) \quad f_{max4} = 0 \cdot psf$

## 2) Assumed Soil Pressure Case 2

Assume triangular distribution with length of bearing ( $L_b$ ) greater than half the major octagon width ( $H_f/2$ ) but less than or equal to difference between the full octagon width ( $H_f$ ) and  $W_e$ .

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2033 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 28354 \cdot ft \cdot k$$

Guess:  $L := \frac{3H_f - 2W_e}{4}$   $f_{max} := 5000 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_5 = 32.4 \text{ ft} \quad f_5 = 3757 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, L_5, 0 \cdot ft \right)$   $L_{b5} = 32.4 \cdot ft$

Maximum soil bearing pressure:  $f_{max5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, f_5, 0 \cdot psf \right)$   $f_{max5} = 3757 \cdot psf$

### 3) Assumed Soil Pressure Case 3

Assume triangular distribution with length of bearing ( $L_b$ ) greater than the difference between the full octagon width ( $H_f$ ) and  $W_e$  but less than the full octagon width ( $H_f$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_W \quad F = 2033 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 28354 \cdot ft \cdot k$$

Guess:  $L := \frac{2H_f - W_e}{2} \quad f_{max} := 4986 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^2 f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^2 f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \quad L_6 = 34.7 \text{ ft} \quad f_6 = 2845 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, L_6, 0 \cdot ft) \quad L_{b6} = 0.0 \cdot ft$

Maximum soil bearing pressure:  $f_{max6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, f_6, 0 \cdot psf) \quad f_{max6} = 0 \cdot psf$

#### 4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing ( $L_b$ ) equal to the full octagon width ( $H_f$ ).

Set  $F=W_W$  and  $M=M_{toe}$ , and solve for  $f_{max}$  and the difference in maximum and minimum bearing pressures ( $df$ ).

$$F := W_W \quad F = 2033 \cdot k \quad M_{toe} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{oW} \quad M_{toe} = 28354 \cdot ft \cdot k$$

Guess:

$$f_{max} := 4611 \cdot psf$$

$$df := 6072 \cdot psf$$

Given

$$F = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\left( \frac{df}{f_{max}} \right) := \text{Find}(df, f_{max})$$

$$f_{max} = 3089 \cdot psf$$

$$df = 4216 \cdot psf$$

If the solution does not converge to the assumed pressure distribution, then the value of bearing length and soil pressure is set to zero.

$$\text{Maximum soil bearing pressure:} \quad f_{max7} := \text{if}(f_{max} - df < 0 \cdot psf, 0 \cdot psf, f_{max}) \quad f_{max7} = 0 \cdot psf$$

$$\text{Minimum soil bearing pressure:} \quad f_{min7} := \text{if}(f_{max7} > 0, f_{max7} - df, 0 \cdot psf) \quad f_{min7} = 0 \cdot psf$$

**D. Bearing Length Check - Extreme Loading**

*Select Bearing Length and Pressure Distribution*

Bearing length:  $L_{bW} := \text{if}(f_{\max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6})$   $L_{bW} = 32.4 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\max W} := \text{if}\left(L_{bW} < \frac{H_f}{2}, f_{\max4}, \text{if}(L_{bW} < H_f - W_e, f_{\max5}, \text{if}(L_{bW} < H_f, f_{\max6}, f_{\max7}))\right)$   
 $f_{\max W} = 3757 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min W} := \text{if}(L_{bW} < H_f, 0 \cdot \text{psf}, f_{\min7})$   $f_{\min W} = 0 \cdot \text{psf}$

Determine controlling load case:  $L_b := L_{bW}$   $L_b = 32.4 \text{ ft}$

Maximum soil bearing pressure:  $f_{\max} := f_{\max W}$   $f_{\max} = 3757 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min} := f_{\min W}$   $f_{\min} = 0 \cdot \text{psf}$

Percent of base by length in  
 compression under extreme loading:  $\frac{L_b}{H_f} = 0.60$

$\text{if}\left(\frac{L_b}{H_f} \geq 0.5, \text{"OK"} , \text{"No Good"}\right) = \text{"OK"}$
--

(Reference 10)

## E. Foundation Volume and Weight Calculations - Normal Loading

Total dead weight:  $W_{\text{totN}} := W_p + W_f + W_s + W_N - F_b$   $W_{\text{totN}} = 2047 \cdot \text{k}$

Overturning moment:  $M_{\text{oN}} := \sqrt{(M_N + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_N = 15197 \cdot \text{k} \cdot \text{ft}$

## F. Soil Pressure Calculations - Normal Loading

### 1) Assumed Soil Pressure Case 1

Assume triangular distribution with length of bearing ( $L_b$ ) greater than  $W_e$  but less than or equal to half the major octagon width ( $H_f/2$ ).

Set  $F=W_{\text{totN}}$  and  $M=M_{\text{toe}}$ , and solve for  $L_b$  and  $f_{\text{max}}$

$F := W_{\text{totN}}$   $F = 2047 \cdot \text{k}$   $M_{\text{toe}} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{\text{oN}}$   $M_{\text{toe}} = 40189 \cdot \text{ft} \cdot \text{k}$

Guess:  $L := \frac{2W_e + H_f}{4}$   $f_{\text{max}} := 3036 \cdot \text{psf}$

Given

$$F = \int_0^{W_e} \left[ f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] dy + \int_{W_e}^L f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy$$

$$M_{\text{toe}} = \int_0^{W_e} \left[ f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left(\frac{y}{W_e} \cdot S_f\right) \right] y dy + \int_{W_e}^L f_{\text{max}} \cdot \left(1 - \frac{y}{L}\right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy$$

$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \text{Find}(L, f_{\text{max}})$   $L_4 = 46.4 \text{ ft}$   $f_4 = 2242 \cdot \text{psf}$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, L_4, 0 \cdot \text{ft} \right)$   $L_{b4} = 0.0 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\text{max}4} := \text{if} \left( L_4 \geq W_e \wedge L_4 \leq \frac{H_f}{2}, f_4, 0 \cdot \text{psf} \right)$   $f_{\text{max}4} = 0 \cdot \text{psf}$



## 2) Assumed Soil Pressure Case 2

Assume triangular distribution with length of bearing ( $L_b$ ) greater than half the major octagon width ( $H_f/2$ ) but less than or equal to difference between the full octagon width ( $H_f$ ) and  $W_e$ .

Set  $F=W_{totN}$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_{totN} \quad F = 2047 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 40189 \cdot ft \cdot k$$

Guess:  $L := \frac{3H_f - 2W_e}{4}$   $f_{max} := 2964 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy$$

$$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := \text{Find}(L, f_{max}) \quad L_5 = 49.1 \text{ ft} \quad f_5 = 2198 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, L_5, 0 \cdot ft \right) \quad L_{b5} = 0.0 \cdot ft$

Maximum soil bearing pressure:  $f_{max5} := \text{if} \left( L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, f_5, 0 \cdot psf \right) \quad f_{max5} = 0 \cdot psf$

### 3) Assumed Soil Pressure Case 3

Assume triangular distribution with length of bearing ( $L_b$ ) greater than the difference between the full octagon width ( $H_f$ ) and  $W_e$  but less than the full octagon width ( $H_f$ ).

Set  $F=W_{totN}$  and  $M=M_{toe}$ , and solve for  $L_b$  and  $f_{max}$

$$F := W_{totN} \quad F = 2047 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 40189 \cdot ft \cdot k$$

Guess:  $L := \frac{2H_f - W_e}{2}$   $f_{max} := 2892 \cdot psf$

Given

$$F = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^L f_{max} \cdot \left( 1 - \frac{y}{L} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \quad L_6 = 49.1 \text{ ft} \quad f_6 = 2197 \cdot psf$$

If the solution does not converge to a bearing length meeting the assumed criteria, then the value of bearing length and soil pressure is set to zero.

Soil bearing length:  $L_{b6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, L_6, 0 \cdot ft)$   $L_{b6} = 49.1 \cdot ft$

Maximum soil bearing pressure:  $f_{max6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, f_6, 0 \cdot psf)$   $f_{max6} = 2197 \cdot psf$

#### 4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing ( $L_b$ ) equal to the full octagon width ( $H_f$ ).

Set  $F=W_{totN}$  and  $M=M_{toe}$ , and solve for  $f_{max}$  and the difference in maximum and minimum bearing pressures ( $df$ ).

$$F := W_{totN} \quad F = 2047 \cdot k \quad M_{toe} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 40189 \cdot ft \cdot k$$

Guess:

$$f_{max} := 2904 \cdot psf$$

$$df := 2657 \cdot psf$$

Given

$$F = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] dy$$

$$M_{toe} = \int_0^{W_e} \left[ \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left( \frac{y}{W_e} \cdot S_f \right) \right] y dy + \int_{W_e}^{\frac{H_f}{2}} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ \frac{2 \cdot W_e \cdot (y - W_e)}{S_f} \right] \right] \cdot y dy \dots$$

$$+ \int_{\frac{H_f}{2}}^{W_e + S_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ S_f + 2 \cdot \left[ W_e - \left[ \frac{2 \cdot W_e \cdot \left( y - \frac{H_f}{2} \right)}{S_f} \right] \right] \right] \cdot y dy + \int_{W_e + S_f}^{H_f} \left( f_{max} - \frac{df \cdot y}{H_f} \right) \cdot \left[ 2 \cdot \left( \frac{S_f}{2} - \frac{S_f}{2} \cdot \frac{y - S_f - W_e}{W_e} \right) \right] \cdot y dy$$

$$\left( \frac{df}{f_{max}} \right) := \text{Find}(df, f_{max}) \quad f_{max} = 2190 \cdot psf \quad df = 2404 \cdot psf$$

If the solution does not converge to the assumed pressure distribution, then the value of bearing length and soil pressure is set to zero.

$$\text{Maximum soil bearing pressure:} \quad f_{max7} := \text{if}(f_{max} - df < 0 \cdot psf, 0 \cdot psf, f_{max}) \quad f_{max7} = 0 \cdot psf$$

$$\text{Minimum soil bearing pressure:} \quad f_{min7} := \text{if}(f_{max7} > 0, f_{max7} - df, 0 \cdot psf) \quad f_{min7} = 0 \cdot psf$$

### **G. Bearing Length Check - Normal Loading**

*Select Bearing Length and Pressure Distribution*

Bearing length:  $L_{bN} := \text{if}(f_{\max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6})$   $L_{bN} = 49.1 \cdot \text{ft}$

Maximum soil bearing pressure:  $f_{\max} := \text{if}\left(L_{bN} < \frac{H_f}{2}, f_{\max4}, \text{if}(L_{bN} < H_f - W_e, f_{\max5}, \text{if}(L_{bN} < H_f, f_{\max6}, f_{\max7}))\right)$   
 $f_{\max} = 2197 \cdot \text{psf}$

Minimum soil bearing pressure:  $f_{\min} := \text{if}(L_{bN} < H_f, 0 \cdot \text{psf}, f_{\min7})$   $f_{\min} = 0 \cdot \text{psf}$

Area of base in compression under normal loading:  $A_N := S_f^2 + \frac{3}{2} \cdot S_f \cdot W_e + (L_{bN} - S_f - W_e) \left[ S_f - \frac{S_f(L_{bN} - S_f - W_e)}{2 \cdot W_e} \right]$   
 $A_N = 2010 \cdot \text{ft}^2$

Percent of base by area in compression under normal loading:  $\frac{A_N}{A} = 97.1 \cdot \%$

$\text{if}\left(\frac{A_N}{A} \geq 0.994, \text{"OK"}, \text{"No Good"}\right) = \text{"No Good"}$	(Reference 10)
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## V. Bearing Capacity Evaluation

(Reference 8)

### A. Design Soil Bearing Pressure - Normal Loading

Design overturning moment:	$M_{dN} := M_{oN}$	$M_{dN} = 15197 \cdot \text{ft} \cdot \text{k}$
Design vertical load:	$V_{dN} := W_{\text{totN}}$	$V_{dN} = 2047 \cdot \text{k}$
Design load eccentricity:	$e_{dN} := \frac{M_{dN}}{V_{dN}}$	$e_{dN} = 7.42 \text{ ft}$
Circular radius of octagon:	$R := \frac{D}{2}$	$R = 25.00 \text{ ft}$
Effective soil area in bearing:	$A_{\text{effN}} := 2 \cdot \left[ \left( R^2 \right) \cdot \text{acos} \left( \frac{e_{dN}}{R} \right) - e_{dN} \cdot \sqrt{R^2 - e_{dN}^2} \right]$	$A_{\text{effN}} = 1232 \text{ ft}^2$
Ellipse soil width in bearing:	$b_{eN} := 2 \cdot (R - e_{dN})$	$b_{eN} = 35.2 \text{ ft}$
Ellipse soil length in bearing:	$l_{eN} := 2 \cdot R \cdot \sqrt{1 - \left( 1 - \frac{b_{eN}}{2 \cdot R} \right)^2}$	$l_{eN} = 47.7 \text{ ft}$
Effective soil length in bearing:	$l_{\text{effN}} := \sqrt{A_{\text{effN}} \cdot \frac{l_{eN}}{b_{eN}}}$	$l_{\text{effN}} = 40.9 \text{ ft}$
Effective soil width in bearing:	$b_{\text{effN}} := \frac{l_{\text{effN}}}{l_{eN}} \cdot b_{eN}$	$b_{\text{effN}} = 30.1 \text{ ft}$
Design bearing pressure:	$f_{dN} := \frac{V_{dN}}{A_{\text{effN}}}$	$f_{dN} = 1661 \cdot \text{psf}$

### B. Bearing Capacity Check - Normal Loading

Allowable bearing pressure:	$f_{\text{all}_N} := 3300 \text{ psf}$	(Reference 2)
Ratio of design bearing pressure to allowable bearing pressure:	$\frac{f_{dN}}{f_{\text{all}_N}} = 0.50$	

### C. Design Soil Bearing Pressure - Extreme Normal Loading

Design overturning moment:  $M_{dW} := \sqrt{(M_e + M_{align} \cdot \cos(\Delta))^2 + (M_{align} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_e = 23813 \cdot \text{k} \cdot \text{ft}$

Design vertical load:  $V_{dW} := W_p + W_f + W_{te} + W_s - F_b$   $V_{dW} = 2044 \cdot \text{k}$

Design load eccentricity:  $e_{dW} := \frac{M_{dW}}{V_{dW}}$   $e_{dW} = 11.65 \text{ ft}$

Circular radius of octagon:  $R = 25.00 \text{ ft}$

Effective soil area in bearing:  $A_{effW} := 2 \cdot \left[ (R^2) \cdot \text{acos}\left(\frac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}^2} \right]$   $A_{effW} = 842 \text{ ft}^2$

Ellipse soil width in bearing:  $b_{eW} := 2 \cdot (R - e_{dW})$   $b_{eW} = 26.7 \text{ ft}$

Ellipse soil length in bearing:  $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eW}}{2 \cdot R}\right)^2}$   $l_{eW} = 44.2 \text{ ft}$

Effective soil length in bearing:  $l_{effW} := \sqrt{A_{effW} \cdot \frac{l_{eW}}{b_{eW}}}$   $l_{effW} = 37.4 \text{ ft}$

Effective soil width in bearing:  $b_{effW} := \frac{l_{effW}}{l_{eW}} \cdot b_{eW}$   $b_{effW} = 22.5 \text{ ft}$

Design bearing pressure:  $f_{dW} := \frac{V_{dW}}{A_{effW}}$   $f_{dW} = 2427 \cdot \text{psf}$

### D. Bearing Capacity Check - Extreme Normal Loading

Allowable bearing pressure:  $f_{allW} := f_{all\_N} \cdot \left(\frac{4}{3}\right) = 4400 \cdot \text{psf}$  (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure:  $\frac{f_{dW}}{f_{allW}} = 0.55$

### E. Design Soil Bearing Pressure - Extreme Abnormal Loading

Design overturning moment:  $M_{dA} := \sqrt{(M_a + M_{align} \cdot \cos(\Delta))^2 + (M_{align} \cdot \sin(\Delta))^2} + (h_b + h_c + h_p) \cdot H_a = 26651 \cdot \text{k} \cdot \text{ft}$

Design vertical load:  $V_{dA} := W_p + W_f + W_{ta} + W_s - F_b \quad V_{dA} = 2033 \cdot \text{k}$

Design load eccentricity:  $e_{dA} := \frac{M_{dA}}{V_{dA}} \quad e_{dA} = 13.11 \text{ ft}$

Circular radius of octagon:  $R = 25.00 \text{ ft}$

Effective soil area in bearing:  $A_{effA} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dA}}{R}\right) - e_{dA} \cdot \sqrt{R^2 - e_{dA}^2} \right] \quad A_{effA} = 715 \text{ ft}^2$

Ellipse soil width in bearing:  $b_{eA} := 2 \cdot (R - e_{dA}) \quad b_{eA} = 23.8 \text{ ft}$

Ellipse soil length in bearing:  $l_{eA} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eA}}{2 \cdot R}\right)^2} \quad l_{eA} = 42.6 \text{ ft}$

Effective soil length in bearing:  $l_{effA} := \sqrt{A_{effA} \cdot \frac{l_{eA}}{b_{eA}}} \quad l_{effA} = 35.8 \text{ ft}$

Effective soil width in bearing:  $b_{effA} := \frac{l_{effA}}{l_{eA}} \cdot b_{eA} \quad b_{effA} = 20.0 \text{ ft}$

Design bearing pressure:  $f_{dA} := \frac{V_{dA}}{A_{effA}} \quad f_{dA} = 2842 \cdot \text{psf}$

### F. Bearing Capacity Check - Extreme Abnormal Loading

Allowable bearing pressure:  $f_{allA} := f_{all\_N} \cdot \left(\frac{4}{3}\right) \cdot \frac{\alpha_c}{\alpha_a} = 5400 \cdot \text{psf} \quad (\text{Reference 2})$

Ratio of design bearing pressure to allowable bearing pressure:  $\frac{f_{dA}}{f_{allA}} = 0.53$

### G. Design Soil Bearing Pressure - Earthquake Loading

Design overturning moment:	$M_{dEQ} := M_{oEQ}$	$M_{dEQ} = 15510 \cdot \text{ft} \cdot \text{k}$
Design vertical load:	$V_{dEQ} := (1 + E_v) \cdot W_{EQ}$	$V_{dEQ} = 2047 \cdot \text{k}$
Design load eccentricity:	$e_{dEQ} := \frac{M_{dEQ}}{V_{dEQ}}$	$e_{dEQ} = 7.58 \text{ ft}$
Circular radius of octagon:	$R = 25.00 \text{ ft}$	
Effective soil area in bearing:	$A_{effEQ} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^2} \right]$	$A_{effEQ} = 1218 \text{ ft}^2$
Ellipse soil width in bearing:	$b_{eEQ} := 2 \cdot (R - e_{dEQ})$	$b_{eEQ} = 34.8 \text{ ft}$
Ellipse soil length in bearing:	$l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eEQ}}{2 \cdot R}\right)^2}$	$l_{eEQ} = 47.6 \text{ ft}$
Effective soil length in bearing:	$l_{effEQ} := \sqrt{A_{effEQ} \cdot \frac{l_{eEQ}}{b_{eEQ}}}$	$l_{effEQ} = 40.8 \text{ ft}$
Effective soil width in bearing:	$b_{effEQ} := \frac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$	$b_{effEQ} = 29.8 \text{ ft}$
Design bearing pressure:	$f_{dEQ} := \frac{V_{dEQ}}{A_{effEQ}}$	$f_{dEQ} = 1681 \cdot \text{psf}$

### H. Bearing Capacity Check - Earthquake Loading

Allowable bearing pressure:	$f_{allEQ} := f_{all\_N} \cdot \left(\frac{4}{3}\right) \cdot \frac{\alpha_e}{\alpha_a} = 5400 \cdot \text{psf}$ (Reference 2)
Ratio of design bearing pressure to allowable bearing pressure:	$\frac{f_{dEQ}}{f_{allEQ}} = 0.31$



## VI. Foundation Stiffness Evaluation - Single Layer Native Soil Sites

Depth of embedment  
(half of foundation embedment):

$$h := \frac{1}{2} \cdot h_c + h_b$$

$h = 45.00 \cdot \text{in}$

Area of the footing:

$$A = 2071 \text{ ft}^2$$

Area of footing in compression:

$$A_N = 2010 \text{ ft}^2$$

Equivalent circular  
radius of footing:

$$R_{\text{stff}} := \sqrt{\frac{A_N}{\pi}}$$

$R_{\text{stff}} = 25.3 \cdot \text{ft}$

Subsoil density:

$$\rho := 125 \cdot \frac{\text{lb}}{\text{ft}^3}$$

(Reference 2)

$$\omega := \rho \cdot g$$

$\omega = 125 \cdot \text{pcf}$

Design shear wave velocity  
for interval from 8 to 50 feet:

$$V_s := 637 \cdot \frac{\text{ft}}{\text{sec}}$$

(Reference 2)

Poisson ratio:

$$\nu := 0.47$$

(Reference 2)

Initial shear modulus:

$$G_o := \rho \cdot V_s^2$$

$G_o = 1576 \cdot \text{ksf}$

$$E_o := 2 \cdot (1 + \nu) \cdot G_o$$

$E_o = 4635 \cdot \text{ksf}$

Initial elastic modulus:

Shear modulus ratio:

$$\gamma := 1 - (1.0) \cdot \left( \frac{f_{dN}}{3 \min(f_{allA}, f_{allW})} \right)^{0.3} = 0.46$$

(Reference 9)

Shear modulus:

$$G := \gamma \cdot G_o$$

$G = 730 \cdot \text{ksf}$

Elastic modulus:

$$E := 2 \cdot (1 + \nu) \cdot G$$

$E = 2146 \cdot \text{ksf}$

Rotational embedment  
Coefficient:

$$\eta_{\psi} := 1 + 1.2 \cdot (1 - \nu) \cdot \frac{h}{R_{stff}} + 0.2 \cdot (2 - \nu) \cdot \left( \frac{h}{R_{stff}} \right)^3 \quad \eta_{\psi} = 1.1 \quad (\text{Reference 5})$$

Embedment coefficient:

$$\eta_x := 1 + 0.55 \cdot (2 - \nu) \cdot \frac{h}{R_{stff}} \quad \eta_x = 1.12 \quad (\text{Reference 5})$$

Rotational stiffness of soil:

$$K_{\psi dyn} := \frac{8 \cdot G \cdot R_{stff}^3}{3 \cdot (1 - \nu)} \cdot \eta_{\psi} \quad K_{\psi dyn} = 88 \cdot \frac{\text{GN} \cdot \text{m}}{\text{rad}} \quad (\text{Reference 5})$$

$$K_{\psi req} = 5.00 \times 10^7 \cdot \frac{\text{kN} \cdot \text{m}}{\text{rad}} \quad (\text{Reference 3})$$

Design check:

$$\frac{K_{\psi dyn}}{K_{\psi req}} = 1.77$$

Translational stiffness  
of foundation:

$$K_{x dyn} := \frac{32 \cdot (1 - \nu) \cdot G \cdot R_{stff}}{7 - 8 \cdot \nu} \cdot \eta_x \quad K_{x dyn} = 1586 \cdot \frac{\text{kN}}{\text{mm}} \quad (\text{Reference 8})$$

Required dynamic translational  
stiffness:

$$K_{x req} = 1.00 \times 10^6 \cdot \frac{\text{kN}}{\text{m}} \quad (\text{Reference 3})$$

Design check:

$$\frac{K_{x dyn}}{K_{x req}} = 1.59$$

## VII. Anchor Bolt Design

### A. Strength Reduction and Load Factors

(Reference 1a and Reference 3)

Normal extreme load factor:  $\alpha_e = 1.35$

Abnormal extreme load factor:  $\alpha_a = 1.10$

Determine controlling extreme load case:  $\alpha_w := \text{if}(\alpha_e \cdot M_e > \alpha_a \cdot M_a, \alpha_e, \alpha_a)$   $\alpha_w = 1.35$

$M := \text{if}(\alpha_e \cdot M_e > \alpha_a \cdot M_a, M_e, M_a)$   $M = 22622 \cdot \text{k} \cdot \text{ft}$

$H := \text{if}(\alpha_e \cdot M_e > \alpha_a \cdot M_a, H_e, H_a)$   $H = 91 \cdot \text{k}$

$W_t := \text{if}(\alpha_e \cdot M_e > \alpha_a \cdot M_a, W_{te}, W_{ta})$   $W_t = 454 \cdot \text{k}$

Beneficial dead load factor:  $\alpha_{d1} := 0.9$

Bearing Factor:  $\phi_{br} \equiv 0.65$

Anchor tension load factor:  $\alpha_{pt} := 1.2$

Fastener Factor:  $\phi_f := 0.75$  (Reference 1c)

Non-Beneficial dead load factor:  $\alpha_{d2} := 1.2$

Shear Factor:  $\phi_v \equiv 0.75$

Earthquake load factor:  $\alpha_{EQ} := 1.0$

Flexure Factor:  $\phi_b \equiv 0.90$

Alignment load factor:  $\alpha_{d3} := 1.0$

Beneficial EQ dead load factor:  $\alpha_{d1EQ} := 0.9 - E_v$   $\alpha_{d1EQ} = 0.90$  (Reference 1)

Non-Beneficial EQ dead load factor:  $\alpha_{d2EQ} := 1.2 + E_v$   $\alpha_{d2EQ} = 1.20$  (Reference 1)

Sagging side load case:  $\alpha_w \text{ Wind} + \alpha_{d1} \text{ Dead}$  where Dead is dead load of soil, concrete, turbine, and tower

Hogging side load case:  $\alpha_{d2} \text{ Dead}$  where Dead is dead load of soil and concrete and uplift edge resistance of soil

## **B. Embedment Ring Dimensions**

Flange width:	$w_{\text{flange}} = 10.94 \cdot \text{in}$	$w_{\text{flange}} = 278 \cdot \text{mm}$
Embedment ring width:	$w := w_{\text{flange}}$	$w = 10.94 \cdot \text{in}$
Embedment ring thickness:	$t := 1.0 \cdot \text{in}$	

## **C. Anchor Bolt Dimensions and Data**

### **ASTMA615 Grade 75**

Nominal anchor bolt diameter:	$d_b := 1.25 \cdot \text{in}$
Bolt area through minimum diameter of threads:	$A_b := 1.27 \cdot \text{in}^2$
Washer diameter.:	$d_n := 3 \cdot \text{in}$
Outside diameter of PVC bolt sleeve:	$d_{\text{SDR}} := 1.660 \text{in}$
Yield strength:	$F_{yb} := 75 \cdot \text{ksi}$
Tensile strength:	$F_t := 100 \cdot \text{ksi}$

## **D. Material Properties**

Concrete strength of pedestal:	$f_{cp} := 5000 \cdot \text{psi}$
Embedment ring yield strength:	$F_y := 36000 \cdot \text{psi}$
Embedment ring tensile strength:	$F_u := 58000 \cdot \text{psi}$

## E. Anchor Bolt Design

Design loss:

$$\mu := 20\%$$

Maximum unfactored moment on bolts:

$$M_{\text{bolt}} := \sqrt{(\max(M_e, M_a) + M_{\text{align}} \cdot \cos(\Delta))^2 + (M_{\text{align}} \cdot \sin(\Delta))^2} = 25844 \cdot \text{k} \cdot \text{ft}$$

Maximum factored moment on bolts:

$$M_{\text{ubolt}} := \sqrt{(\alpha_w \cdot M + \alpha_{d3} \cdot M_{\text{align}} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{\text{align}} \cdot \sin(\Delta))^2} = 31047 \cdot \text{k} \cdot \text{ft}$$

Maximum factored seismic moment on bolts:

$$M_{\text{usbolt}} := \sqrt{(\alpha_{EQ} \cdot M_{OE} + \alpha_{d3} \cdot M_{\text{align}} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{\text{align}} \cdot \sin(\Delta))^2} = 15017 \cdot \text{k} \cdot \text{ft}$$

Minimum pre-tension for wind loading (if used):

$$T_{\text{preW}} := \left( \frac{4 \cdot M_{\text{bolt}}}{N \cdot D_{\text{avg}}} - \frac{W_t}{N} \right) \cdot (1 + \mu) \quad T_{\text{preW}} = 59.2 \cdot \text{k}$$

Minimum pre-tension for fatigue loading:

$$T_{\text{preFAT}} := \left( \frac{4 \cdot \max(M_{\text{maxnorth}})}{N \cdot D_{\text{avg}}} - \frac{W_{\text{mean}}}{N} \right) \cdot (1 + \mu) \quad T_{\text{preFAT}} = 42.9 \cdot \text{k}$$

Set to  $T_{\text{pre}}$ :

$$T_{\text{pre}} := 72 \cdot \text{k}$$

$$\% \text{yield} := \frac{T_{\text{pre}}}{F_{yb} \cdot A_b} \quad \% \text{yield} = 76\%$$

$$\% \text{ultimate} := \frac{T_{\text{pre}}}{F_t \cdot A_b} \quad \% \text{ultimate} = 57\%$$

Wind load maximum factored tension load in anchor:

$$T_{\text{uW}} := \frac{4 \cdot M_{\text{ubolt}}}{N \cdot D_{\text{avg}}} - \frac{\alpha_{d1} \cdot W_t}{N} \quad T_{\text{uW}} = 60 \cdot \text{k}$$

Seismic load maximum factored tension load in anchor:

$$T_{\text{uEQ}} := \frac{4 \cdot M_{\text{usbolt}}}{N \cdot D_{\text{avg}}} - \frac{\alpha_{d1EQ} \cdot W_{OE}}{N} \quad T_{\text{uEQ}} = 28 \cdot \text{k}$$

Fatigue load maximum tension load in anchor:

$$T_{\text{uFAT}} := \frac{4 \cdot \max(M_{\text{maxnorth}})}{N \cdot D_{\text{avg}}} - \frac{W_{\text{mean}}}{N} \quad T_{\text{uFAT}} = 36 \cdot \text{k}$$

Determine controlling load case:

$$T_u := \max(T_{\text{uW}}, T_{\text{uEQ}}, T_{\text{uFAT}}) \quad T_u = 60 \cdot \text{kip}$$

Design tension strength:

$$\phi T_n := \min(\phi_F \cdot F_t \cdot A_b, \phi_b \cdot F_{yb} \cdot A_b) \quad \phi T_n = 86 \cdot \text{k} \quad (\text{Reference 1c})$$

Design check:

$$\frac{T_u}{\phi T_n} = 0.70 \quad \text{Shear stress in bolt is negligible and, therefore, is not included.}$$

## VIII. Bottom Flange Bearing, Grout, and Embedment Plate Connection Design

### A. Material Properties

3-day grout strength:  $f_{c3} := 4500 \cdot \text{psi}$

28-day grout strength:  $f_{c28} := 8000 \cdot \text{psi}$

Grout thickness:  $t_{gr} := 0 \cdot \text{in}$

Bearing area of base flange:  $A_{br1} := \frac{\pi}{4} \cdot (OD^2 - ID^2) - N \cdot \frac{\pi \cdot d_{SDR}^2}{4}$   $A_{br1} = 5488 \cdot \text{in}^2$

Section modulus of base flange:  $S_1 := \frac{\pi}{32 \cdot OD} (OD^4 - ID^4) \dots$

$$+ \left[ \frac{2}{D_o} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \dots \right. \right. \right. \right.$$

$$\left. \left. \left. + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right]^2 \right]$$

$S_1 = 217587 \cdot \text{in}^3$

Area at bottom of grout:  $A_1 := w_{flange}$   $A_1 = 10.9 \cdot \text{in}$

Area of limiting bearing within grout:  $A_2 := w_{flange} + 2 \cdot t_{gr}$   $A_2 = 10.9 \cdot \text{in}$

$A := \min \left( \sqrt{\frac{A_2}{A_1}}, 2 \right)$   $A = 1.00$

### B. Check 3 Day Grout Strength

Design bearing strength:  $\phi b_{n3} := \phi_{br} \cdot 0.85 \cdot f_{c3} \cdot A$  (Reference 1a)  $\phi b_{n3} = 2.5 \cdot \text{ksi}$

Ultimate self weight bearing stress:  $b_{u3D} := \left( \alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{u3D} = 2.3 \cdot \text{ksi}$

Ultimate seismic bearing stress:  $b_{u3EQ} := \left( \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{u3EQ} = 2.3 \cdot \text{ksi}$

Determine controlling load case and check capacity:  $b_{u3} := \max(b_{u3D}, b_{u3EQ})$   $b_{u3} = 2.3 \cdot \text{ksi}$

$\frac{b_{u3}}{\phi b_{n3}} = 0.93$
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### C. Check 28 Day Grout Strength

Design bearing strength:  $\phi b_{n28} := \phi_{br} \cdot 0.85 \cdot f_{c28} \cdot A$  (Reference 1a)  $\phi b_{n28} = 4.4 \cdot \text{ksi}$

Ultimate wind bearing stress:  $b_{u28W} := \alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \frac{M_{ubolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$   $b_{u28W} = 4.02 \cdot \text{ksi}$

Ultimate seismic bearing stress:  $b_{u28EQ} := \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{EQ} \cdot \frac{M_{usbolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$   $b_{u28EQ} = 3.1 \cdot \text{ksi}$

Determine controlling load case and check capacity:  $b_{u28} := \max(b_{u28W}, b_{u28EQ})$   $b_{u28} = 4.0 \cdot \text{ksi}$

$\frac{b_{u28}}{\phi b_{n28}} = 0.91$
---------------------------------------

### D. Check Bottom Flange Bearing on Concrete

Grout thickness:

$$t_g := 0 \cdot \text{in}$$

Pullout force due to wind:

$$P_{uW} := 2 \left( \frac{4M_{ubolt}}{N \cdot D_{avg}} - \alpha_{d1} \cdot \frac{W_t}{N} \right) \quad P_{uW} = 121 \cdot \text{k}$$

Pullout force due to seismic:

$$P_{uEQ} := 2 \left( \frac{4 \cdot M_{usbolt}}{N \cdot D_{avg}} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{N} \right) \quad P_{uEQ} = 55 \cdot \text{k}$$

Determine controlling load case:


$$P_u := \max(P_{uW}, P_{uEQ}) \quad P_u = 121 \cdot \text{k}$$

Bearing area at bottom of grout:

$$A_{br2} := \frac{\pi}{4} \cdot \left[ (OD + 2 \cdot t_g)^2 - (ID - 2 \cdot t_g)^2 \right] - N \cdot \frac{\pi \cdot d_{SDR}^2}{4} \quad A_{br2} = 5488 \cdot \text{in}^2$$

Section modulus at bottom of grout:

$$S_2 := \frac{\pi}{32 \cdot (OD + 2 \cdot t_g)} \left[ (OD + 2 \cdot t_g)^4 - (ID - 2 \cdot t_g)^4 \right] \dots$$

$$+ \left[ \frac{2}{(OD + 2 \cdot t_g)} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \dots \right. \right. \right. \right. \left. \left. \left. + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \right]$$


$$S_2 = 217939 \cdot \text{in}^3$$

Ultimate wind stress:

$$b_{uW} := \frac{M_{ubolt}}{S_2} + \alpha_{d2} \cdot \frac{W_t}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}} \quad b_{uW} = 4.0 \cdot \text{ksi}$$

Ultimate seismic stress:

$$b_{uEQ} := \frac{M_{usbolt}}{S_2} + \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}} \quad b_{uEQ} = 3.1 \cdot \text{ksi}$$

Determine controlling load case:

$$b_u := \max(b_{uW}, b_{uEQ}) \quad b_u = 4.0 \cdot \text{ksi}$$

Check Bearing Plate Stresses on Concrete Due to Pre-tension and Extreme Wind Force:

Compute Areas being loaded: (Reference 1a)

Area at bottom of grout:

$$A_1 := w_{flange} + 2 \cdot t_g \quad A_1 = 10.9 \cdot \text{in}$$

Area of limiting bearing within concrete:

$$A_2 := A_1 + [C - (OD + 2 \cdot t_g)] \quad A_2 = 35.6 \cdot \text{in}$$

$$A := \min \left( \sqrt{\frac{A_2}{A_1}}, 2 \right) \quad A = 1.80$$

Design bearing strength:

$$\phi b_n := \phi_{br} \cdot 0.85 \cdot f_{cp} \cdot A \quad (\text{Reference 1a}) \quad \phi b_n = 5.0 \cdot \text{ksi}$$

$$\frac{b_u}{\phi b_n} = 0.81$$



**E. Check Pullout Strength of Embedment Ring/Anchor Bolt Connection**

Conservatively ignored

**E2. Check Pullout Strength of Embedment Ring/Anchor Bolt Connection (concrete only)**

Due to Wind:  $P_{uW} = 120.57 \cdot k$

Angle of subtended arc:  $\alpha_1 := \frac{4 \cdot \pi}{N}$   $\alpha_1 = 5.14 \cdot \text{deg}$

Depth of Outer Shear Cone:  $l_{eo} := \frac{h_b + h_c - h_e - (B - D_{avg} - w) \cdot \frac{1}{2}}{\frac{h_c}{a} + 1} \dots$   $l_{eo} = 49.2 \cdot \text{in}$   
 $+ (B - D_{avg} - w) \cdot \frac{1}{2}$

Area of Outer Shear Cone Segment:  $A_o := \frac{\alpha_1}{360 \cdot \text{deg}} \cdot \pi \cdot l_{eo} \cdot \sqrt{2} \cdot (OD + l_{eo})$   $A_o = 713.33 \cdot \text{in}^2$

Area of Bottom Plane:  $A_{bot} := \frac{\alpha_1}{360 \cdot \text{deg}} \cdot \pi \cdot \frac{ID^2}{4}$   $A_{bot} = 278.26 \cdot \text{in}^2$

Total Areas:  $A_{conc} := A_o + A_{bot}$   $A_{conc} = 991.59 \cdot \text{in}^2$

Pullout Strength of Outer Shear Cone Segment:  $\phi V_{n_p} := \phi_v \cdot A_o \cdot 2.8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \dots$   $\phi V_{n_p} = 164.95 \cdot \text{kip}$   
 $+ \phi_v \cdot A_{bot} \cdot 4 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c}$

$\frac{P_{uW}}{\phi V_{n_p}} = 0.73$
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## F. Check Bending Strength of Embedment Plate

Nut to nut circumferential distance:  $d_1 := \frac{2 \cdot D_o \cdot \pi}{N} - d_n$   $d_1 = 4.82 \cdot \text{in}$

Nut to nut radial distance:  $d_2 := \left( \frac{D_o - D_i}{2} \right) - d_n$   $d_2 = 2.83 \cdot \text{in}$

Edge distance:  $d_3 := \left( \frac{OD - D_o}{2} \right) - \frac{d_n}{2}$   $d_3 = 1.06 \cdot \text{in}$

Ultimate wind stress:  $b_{\text{uplate}W} := \frac{M_{\text{ubolt}}}{S_1} - \alpha_{d1} \cdot \frac{W_t}{A_{br1}}$   $b_{\text{uplate}W} = 1.6 \cdot \text{ksi}$

Ultimate seismic stress:  $b_{\text{uplate}EQ} := \frac{M_{\text{usbolt}}}{S_1} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{A_{br1}}$   $b_{\text{uplate}EQ} = 0.8 \cdot \text{ksi}$

Determine controlling load case:  $b_{\text{uplate}} := \max \left( b_{\text{uplate}W}, b_{\text{uplate}EQ}, \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$   $b_{\text{uplate}} = 2.2 \cdot \text{ksi}$

Plastic section modulus per inch:  $Z_y := \frac{t^2}{4}$   $Z_y = 0.25 \cdot \frac{\text{in}^3}{\text{in}}$

Section modulus per inch:  $S_y := \frac{t^2}{6}$   $S_y = 0.17 \cdot \frac{\text{in}^3}{\text{in}}$

Check circumferential nut to nut bending

$M_{u.e1} := b_{\text{uplate}} \cdot \frac{d_1^2}{12} = 4.27 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$   $\phi M_n := \phi_b \cdot \min(F_y \cdot Z_y, 1.6 \cdot F_y \cdot S_y) = 8.10 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e1}}{\phi M_n} = 0.53$

Check circumferential nut to nut bending at splice

$M_{u.e2} := b_{\text{uplate}} \cdot \frac{d_1^2}{8} = 6.40 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e2}}{\phi M_n} = 0.79$

Check radial nut to nut bending

$M_{u.e3} := b_{\text{uplate}} \cdot \frac{d_2^2}{12} = 1.47 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e3}}{\phi M_n} = 0.18$

Check nut to edge bending

$M_{u.e4} := \frac{b_{\text{uplate}}}{2} \cdot d_3^2 = 1.24 \cdot \frac{\text{in} \cdot \text{k}}{\text{in}}$  (Reference 1c)  $\frac{M_{u.e4}}{\phi M_n} = 0.15$

Check shear rupture of washer through plate

$V_{uj} := \max \left( \alpha_{pt} \cdot T_{pre}, \frac{P_u}{2} \right) = 86.40 \cdot \text{k}$   $\phi V_n := \phi_v \cdot \pi \cdot d_n \cdot t \cdot 0.6 \cdot F_u = 245.99 \cdot \text{k}$  (Reference 1c)  $\frac{V_{uj}}{\phi V_n} = 0.35$

## IX-a. Concrete Design - Extreme Loads

### A. Design Functions

Function describing the volume of concrete for each slice of the moment/shear calculations.

$$\text{ConcreteVolume}(y) := \begin{cases} h_b \cdot (B + 2 \cdot y) + \frac{y}{a} \cdot h_c \cdot (B + y) & \text{if } y \leq a \\ h_b \cdot (D) + h_c \cdot (B + a) & \text{otherwise} \end{cases}$$

Functions describing the weight of the soil wedge pieces acting on each slice of the moment/shear calculations.

$$\text{StaticSoilWedgeWeight}(\gamma_{sd}, \gamma_{ss}) := \begin{cases} \gamma_{sd} \cdot \frac{B \cdot \tan(\theta)}{2} \cdot (h_s - h_b)^2 & \text{if } d_{GWT} \geq h_s - h_b \\ \frac{B \cdot \tan(\theta)}{2} \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{otherwise} \end{cases}$$

$$\text{VariableSoilWedgeWeight}(y, \gamma_{sd}, \gamma_{ss}) := \begin{cases} 0 & \text{if } d_{GWT} \geq h_s - h_b \\ \text{otherwise} \\ \begin{cases} \sqrt{2} \cdot \tan(\theta) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{if } y \leq a \\ \tan(\theta) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWT})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWT})^2 \right] \right] & \text{otherwise} \end{cases} \end{cases}$$

Function describing the volume of dry soil over each slice of the moment/shear calculations.

$$\text{DrySoilVolume}(h_j, y) := \begin{cases} \text{if } d_{\text{GWT}} \geq h_s - h_b \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \frac{y^2 \cdot h_c}{a} + \sqrt{2} \cdot \tan(\theta) \cdot (h_s - h_b)^2 \right] & \text{if } y \leq a \\ D \cdot \left[ (h_s - h_b) - h_c \right] + h_c \cdot a + \tan(\theta) \cdot (h_s - h_b)^2 & \text{otherwise} \\ \text{if } d_{\text{GWT}} \leq h_s - h_j \\ d_{\text{GWT}} \cdot (B + 2 \cdot y) & \text{if } y \leq a \\ D \cdot d_{\text{GWT}} & \text{otherwise} \\ \text{otherwise} \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \left[ \frac{y^2 \cdot h_c}{a} - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 \right] \right] & \text{if } y \leq a \\ \left[ D \cdot (h_s - h_b - h_c) + \left[ h_c \cdot a - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 \right] \right] & \text{otherwise} \end{cases}$$

Function describing the volume of saturated soil over each slice of the moment/ shear calculations.

$$\text{SaturatedSoilVolume}(h_j, y) := \begin{cases} 0 & \text{if } d_{\text{GWT}} \geq h_s - h_b \\ \text{if } d_{\text{GWT}} \leq h_s - h_j \\ \left( (B + 2 \cdot y) \cdot \left( h_s - h_b - \frac{y}{a} \cdot h_c - d_{\text{GWT}} \right) + \frac{y^2 \cdot h_c}{a} \right) & \text{if } y \leq a \\ (h_s - h_b - h_c - d_{\text{GWT}}) \cdot D + h_c \cdot a & \text{otherwise} \\ \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWT}})^2 & \text{otherwise} \end{cases}$$

Function describing the effect of groundwater on the material weights over each slice of the moment/ shear calculations.

$$\text{BuoyancyWeight}(y) := \begin{cases} 0 & \text{if } d_{\text{GWT}} \geq h_s \\ \text{if } d_{\text{GWT}} < h_s \\ (B + 2 \cdot y) \cdot (h_s - d_{\text{GWT}}) & \text{if } y \leq a \\ (h_s - d_{\text{GWT}}) \cdot D & \text{otherwise} \end{cases}$$

## B. Design Soil Bearing Pressure Wind Loading

(Reference 8)

Design overturning moment:  $M_{dW} := \sqrt{(\alpha_w \cdot M + \alpha_{d3} \cdot M_{align} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{align} \cdot \sin(\Delta))^2} + \alpha_w \cdot (h_b + h_c + h_p) \cdot H$

$$M_{dW} = 31968 \cdot \text{k} \cdot \text{ft}$$

Design vertical load:  $V_{dW} := \alpha_{d1} \cdot \left( W_p + W_f + W_s - \frac{F_b}{\alpha_{d1}} + W_t \right)$

$$V_{dW} = 1790 \cdot \text{k}$$

Design load eccentricity:  $e_{dW} := \frac{M_{dW}}{V_{dW}}$

$$e_{dW} = 17.86 \text{ ft}$$

Circular radius of octagon:  $R := \frac{D}{2}$

$$R = 25.00 \text{ ft}$$

Effective soil area in bearing:  $A_{effW} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}^2} \right]$

$$A_{effW} = 344 \cdot \text{ft}^2$$

Ellipse soil width in bearing:  $b_{eW} := 2 \cdot (R - e_{dW})$

$$b_{eW} = 14.3 \text{ ft}$$

Ellipse soil length in bearing:  $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eW}}{2 \cdot R}\right)^2}$

$$l_{eW} = 35.0 \text{ ft}$$

Effective soil length in bearing:  $l_{effW} := \sqrt{A_{effW} \cdot \frac{l_{eW}}{b_{eW}}}$

$$l_{effW} = 29.0 \text{ ft}$$

Design bearing pressure:  $f_{dW} := \frac{V_{dW}}{A_{effW}}$

$$f_{dW} = 5204 \cdot \text{psf}$$

Effective soil width in bearing:  $b_{effW} := \frac{l_{effW}}{l_{eW}} \cdot b_{eW}$

$$b_{effW} = 11.8 \text{ ft}$$

$$x_{startW} := \frac{D}{2} - e_{dW} - \frac{b_{effW}}{2}$$

$$x_{startW} = 1.22 \text{ ft}$$

### C. Design Soil Bearing Pressure Seismic Loading

(Reference 8)

Design overturning moment: 
$$M_{dEQ} := \sqrt{(\alpha_{EQ} \cdot M_{OE} + \alpha_{d3} \cdot M_{align} \cdot \cos(\Delta))^2 + (\alpha_{d3} \cdot M_{align} \cdot \sin(\Delta))^2} + \alpha_{EQ} \cdot (h_b + h_c + h_p) \cdot H_{OE}$$

$M_{dEQ} = 15510 \cdot \text{ft} \cdot \text{k}$

Design vertical load: 
$$V_{dEQ} := \alpha_{d1EQ} \cdot \left( W_p + W_f + W_s - \frac{F_b}{\alpha_{d1EQ}} + W_{OE} \right)$$

$V_{dEQ} = 1792 \cdot \text{kip}$

Design load eccentricity: 
$$e_{dEQ} := \frac{M_{dEQ}}{V_{dEQ}}$$

$e_{dEQ} = 8.65 \text{ ft}$

Effective soil area in bearing: 
$$A_{effEQ} := 2 \cdot \left[ (R^2) \cdot \arccos\left(\frac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^2} \right]$$

$A_{effEQ} = 1116 \cdot \text{ft}^2$

Ellipse soil width in bearing: 
$$b_{eEQ} := 2 \cdot (R - e_{dEQ})$$

$b_{eEQ} = 32.7 \text{ ft}$

Ellipse soil length in bearing: 
$$l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eEQ}}{2 \cdot R}\right)^2}$$

$l_{eEQ} = 46.9 \text{ ft}$

Effective soil length in bearing: 
$$l_{effEQ} := \sqrt{A_{effEQ} \cdot \frac{l_{eEQ}}{b_{eEQ}}}$$

$l_{effEQ} = 40.0 \text{ ft}$

Design bearing pressure: 
$$f_{dEQ} := \frac{V_{dEQ}}{A_{effEQ}}$$

$f_{dEQ} = 1606 \cdot \text{psf}$

Effective soil width in bearing: 
$$b_{effEQ} := \frac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$$

$b_{effEQ} = 27.9 \text{ ft}$

$$x_{startEQ} := \frac{D}{2} - e_{dEQ} - \frac{b_{effEQ}}{2}$$

$x_{startEQ} = 2.40 \text{ ft}$

## D. Structural Calculations

Area of pedestal:  $A_{ped} := \pi \cdot \frac{C^2}{4}$   $A_{ped} = 227 \cdot ft^2$

Equivalent square dimension:  $S_{ped} := \sqrt{A_{ped}}$   $S_{ped} = 15.1 \text{ ft}$

Distance to critical section:  $x_{face} := \frac{D - S_{ped}}{2}$   $x_{face} = 17.5 \text{ ft}$

Number of section slices to be taken within "a":  $n := \text{trunc}\left(\frac{a}{ft} \cdot 2\right)$   $n = 29$

Number of section slices to be taken beyond "a":  $n_j := \text{trunc}\left(\frac{x_{face}}{ft} \cdot 2\right) - n$   $n_j = 5.00$

Sloped portion of footing:  $i := 1, 2 .. n$

From a to the critical section ( $x_{face}$ ):  $j := n + 1, n + 2 .. n + n_j$

Array counter for all slices:  $q := 1, 2 .. n + n_j$

Plan location of section:  $x_i := \frac{i}{2} \cdot ft$

$$x_j := \max(x) + (x_{face} - \max(x)) \cdot \frac{j - n}{n_j}$$

Height of section:  $h_i := h_b + \frac{x_i}{a} \cdot h_c$   $h_j := h_b + h_c$

Depth, d, as a function of distance along the sloped portion of the foundation is:  $d_i := h_b + \frac{x_i}{a} \cdot h_c - 3.75 \cdot \text{in}$

$$d_j := h_b + h_c - 3.75 \cdot \text{in}$$

Equivalent square dimension of embedment ring:  $S_{embedring} := \sqrt{\pi \cdot \frac{OD^2}{4}}$   $S_{embedring} = 13.2 \text{ ft}$

Alternate distance to critical section based on edge of embedment ring:  $x_{face\_alt} := \frac{D - S_{embedring}}{2}$   $x_{face\_alt} = 18.4 \text{ ft}$

Plan location (alternate) of section:  $x_{alt_i} := \frac{i}{2} \cdot ft$

$$x_{alt_j} := \max(x_{alt}) + (x_{face\_alt} - \max(x_{alt})) \cdot \frac{j - n}{n_j}$$

The exact solution for the factored shear force under wind loading due to soil bearing pressure along the sloped portion of the foundation is:

Design Shear from edge of footing to just before  $b_{effW}$

$$V_{uW_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startW} \\ \int_{x_{startW}}^{\min(x_i, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \, dy & \text{otherwise} \end{cases} + \int_0^{x_i} -\alpha_{d1} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{array} \right) dy \dots$$

$$+ -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w)$$

Design Shear from a to  $x_{face}$

$$V_{uW_j} := \int_{x_{startW}}^{\min(x_j, x_{startW} + b_{effW})} f_{dW} \cdot l_{effW} \, dy + \int_0^a -\alpha_{d1} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{array} \right) dy \dots$$

$$+ \int_a^{x_j} -\alpha_{d1} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{array} \right) dy + \left( -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \dots \right. \\ \left. + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \right)$$



The exact solution for the factored shear force under seismic loading due to soil bearing pressure along the sloped portion of the foundation is:

Design Shear from edge of footing to just before  $b_{effEQ}$

$$V_{uEQ_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startEQ} \\ \int_{x_{startEQ}}^{\min(x_i, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ} \, dy & \text{otherwise} \end{cases} + \int_0^{x_i} -\alpha_{d1EQ} \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy \dots$$

$$+ -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w)$$

Design Shear from a to  $x_{face}$

$$V_{uEQ_j} := \int_{x_{startEQ}}^{\min(x_j, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ} \, dy + \int_0^a -\alpha_{d1EQ} \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy \dots$$

$$+ \int_a^{x_j} -\alpha_{d1EQ} \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ - \text{BuoyancyWeight}(y) \cdot \gamma_w \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \end{array} \right) dy + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \dots$$

$$+ \text{StaticSoilWedgeWeight}(0pcf, \gamma_w)$$



*Bottom Reinforcing Design Moments*

Solution for the design bending moment due to soil bearing pressure under wind loading is:

$$\begin{aligned}
 M_{ubot1W_i} := & \begin{cases} 0 & \text{if } x_i \leq x_{startW} \\ \int_{x_{startW}}^{\min(x_i, x_{startW}+b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_i - y) \, dy & \text{otherwise} \end{cases} + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_i \dots \\
 & + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_i \dots \\
 & + \int_{x_i}^{x_i} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_i - y) \, dy \right) \\
 M_{ubot1W_j} := & \int_{x_{startW}}^{\min(x_j, x_{startW}+b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_j - y) \, dy + \int_0^a \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_j - y) \, dy \dots \right) \\
 & + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_j + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_j \dots \\
 & + \int_a^{x_j} \left( -\alpha_{d1} \cdot \left( \begin{aligned} & \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ & - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \dots \\ & + \frac{\alpha_{d1}}{\alpha_{d1}} \dots \\ & + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{aligned} \right) \cdot (x_j - y) \, dy \right)
 \end{aligned}$$

Solution for the design bending moment due to soil bearing pressure under seismic loading is:

$$M_{ubot1EQ_i} := \begin{cases} 0 & \text{if } x_i \leq x_{startEQ} \dots \\ \int_{x_{startEQ}}^{\min(x_i, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ}(x_i - y) dy & \text{otherwise} \\ + \int_0^{x_i} \left( -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_i - y) dy \dots \\ + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_i \dots \end{cases}$$

$$M_{ubot1EQ_j} := \begin{cases} \int_{x_{startEQ}}^{\min(x_j, x_{startEQ} + b_{effEQ})} f_{dEQ} \cdot l_{effEQ}(x_j - y) dy \dots \\ + \int_0^a \left( -\alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_j - y) dy \dots \\ + -\alpha_{d1EQ} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_j \dots \\ + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_j \dots \\ + - \int_0^{x_j} \left( \alpha_{d1EQ} \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \dots \\ - \text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w) \\ + \frac{\alpha_{d1EQ}}{\alpha_{d1EQ}} \dots \\ + \frac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_j - y) dy \end{cases}$$

Determine controlling load case for bottom moments:  $^c M_{ubot1}_q := \max(M_{ubot1W}_q, M_{ubot1EQ}_q)$

*Top Reinforcing Design Moments*

The solution for the design bending moment due to the weight of concrete and soil above the footing and soil resistance along edge of footing is:

$$M_{utop1_i} := \int_0^{x_{alt_i}} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_i} - y) dy \dots$$

$$+ \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \text{StaticSoilWedgeWeight}(\gamma_{sstop}, \gamma_{sstop}) \cdot x_{alt_i}$$

$$M_{utop1_j} := \int_0^a \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_j} - y) dy \dots$$

$$+ \int_a^{x_{alt_j}} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \left( \begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \dots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \dots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sstop}, \gamma_{sstop}) \end{array} \right) \cdot (x_{alt_j} - y) dy \dots$$



### **E. Top and Bottom Reinforcing Selection**

Top middle bars:                       $\text{Size}_{\text{tmb}} := 6$                        $s_{\text{topm}} := 8 \cdot \text{in}$

Top outside bars:                       $\text{Size}_{\text{tob}} := 6$                        $s_{\text{topo}} := 8 \cdot \text{in}$

Bottom middle bars:                       $\text{Size}_{\text{bmb}} := 11$                        $s_{\text{botm}} := 9 \cdot \text{in}$

Bottom outside bars:                       $\text{Size}_{\text{bob}} := 7$                        $s_{\text{boto}} := 9 \text{in}$

Distance from centerline that defines  
middle/outside boundary:                       $W_m := \frac{C}{2} + 1.5 \cdot (h_b + h_c)$                        $W_m = 16.25 \text{ ft}$

### **F. Bar Cutoff Locations**

Distance of top bar cutoff from  
edge of footing:                       $cd_{\text{top}} := 5 \cdot \text{ft}$

Distance of bottom bar cutoff from  
edge of footing:                       $cd_{\text{bot}} := 0 \cdot \text{ft}$



Assign properties using lookup function, depending on bar size.

$$d_{i_{topm}} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{topm}} = 0.750 \cdot \text{in}$$

$$d_{i_{topo}} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{topo}} = 0.750 \cdot \text{in}$$

$$d_{i_{botm}} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{botm}} = 1.410 \cdot \text{in}$$

$$d_{i_{boto}} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 1) \cdot \text{in} \quad d_{i_{boto}} = 0.875 \cdot \text{in}$$

$$A_{topm} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{topm} = 0.44 \cdot \text{in}^2$$

$$A_{topo} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{topo} = 0.44 \cdot \text{in}^2$$

$$A_{botm} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{botm} = 1.56 \cdot \text{in}^2$$

$$A_{boto} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 2) \cdot \text{in}^2 \quad A_{boto} = 0.60 \cdot \text{in}^2$$

$$W_{topm} := \text{vlookup}(\text{Size}_{tmb}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{topm} = 1.502 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{topo} := \text{vlookup}(\text{Size}_{tob}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{topo} = 1.502 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{botm} := \text{vlookup}(\text{Size}_{bmb}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{botm} = 5.313 \cdot \frac{\text{lbf}}{\text{ft}}$$

$$W_{boto} := \text{vlookup}(\text{Size}_{bob}, \text{ACI\_bar\_table}, 3) \cdot \text{lbf} \div \text{ft} \quad W_{boto} = 2.044 \cdot \frac{\text{lbf}}{\text{ft}}$$

## **G. Bottom Reinforcing Development Length Past Critical Section**

### 1) Middle Bars

Rebar yield strength:  $f_{yB} = 75000 \cdot \text{psi}$

28 day concrete strength:  $f_c = 5000 \cdot \text{psi}$

Bottom middle layer bar spacing:  $s_{\text{botm}} = 9.00 \cdot \text{in}$

Bottom middle layer bar diameter:  $d_{i\text{botm}} = 1.410 \cdot \text{in}$

Reinforcement location factor:  $\alpha := 1.0$  <12 inches of concrete cast below (Reference 1a)

Coating factor:  $\beta := 1.0$  uncoated

Reinforcement size factor:  $\gamma := \text{if}(d_{i\text{botm}} < 0.875 \cdot \text{in}, 0.8, 1.0)$   $\gamma = 1.0$

Lightweight concrete factor:  $\lambda := 1.0$  normal weight concrete

Spacing factor:  $c := \min\left(\frac{\min\left(\frac{s_{\text{botm}}}{2}, c_{\text{cbot}} + \frac{d_{i\text{botm}}}{2}\right)}{d_{i\text{botm}}}, 2.5\right)$   $c = 2.5$

Tension development length past critical section:  $l_{\text{dbotm}} := \frac{3 \cdot f_{yB} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c}} \cdot d_{i\text{botm}}$   $l_{\text{dbotm}} = 45 \cdot \text{in}$  (Reference 1a)

### 2) Outside Bars

Bottom outside layer bar spacing:  $s_{\text{boto}} = 9.00 \cdot \text{in}$

Bottom outside layer bar diameter:  $d_{i\text{boto}} = 0.875 \cdot \text{in}$

Reinforcement size factor:  $\gamma := \text{if}(d_{i\text{boto}} < 0.875 \cdot \text{in}, 0.8, 1.0)$   $\gamma = 1.0$

Spacing factor:  $c := \min\left(\frac{\min\left(\frac{s_{\text{boto}}}{2}, c_{\text{cbot}} + \frac{d_{i\text{boto}}}{2}\right)}{d_{i\text{boto}}}, 2.5\right)$   $c = 2.5$

Tension development length past critical section:  $l_{\text{dboto}} := \frac{3 \cdot f_{yB} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c}} \cdot d_{i\text{boto}}$   $l_{\text{dboto}} = 28 \cdot \text{in}$  (Reference 1a)

## H. Top Reinforcing Development Length Past Critical Section

### 1) Middle Bars

Top middle layer bar spacing:	$s_{topm} = 8.00 \cdot \text{in}$		
Top middle layer bar size:	$d_{i_{topm}} = 0.750 \cdot \text{in}$		
Reinforcement location factor:	$\alpha := 1.3$	>12 inches of concrete cast below	(Reference 1a)
Coating factor:	$\beta := 1.0$	uncoated	
Reinforcement size factor:	$\gamma := \text{if}(d_{i_{topm}} < 0.875 \cdot \text{in}, 0.8, 1.0)$	$\gamma = 0.8$	
Lightweight concrete factor:	$\lambda := 1.0$	normal weight concrete	
Spacing factor:	$c := \min\left(\frac{\min\left(\frac{s_{topm}}{2}, cc_{top} + \frac{d_{i_{topm}}}{2}\right)}{d_{i_{topm}}}, 2.5\right)$	$c = 2.5$	
Tension development length past critical section:	$l_{d_{topm}} := \frac{3 \cdot f_{yT} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c \cdot \text{psi}}} \cdot d_{i_{topm}}$	$l_{d_{topm}} = 25 \cdot \text{in}$	(Reference 1a)

### 2) Outside Bars

Top outside layer bar spacing:	$s_{topo} = 8.00 \cdot \text{in}$		
Top outside layer bar size:	$d_{i_{topo}} = 0.750 \cdot \text{in}$		
Reinforcement size factor:	$\gamma := \text{if}(d_{i_{topo}} < 0.875 \cdot \text{in}, 0.8, 1.0)$	$\gamma = 0.8$	
Spacing factor:	$c := \min\left(\frac{\min\left(\frac{s_{topo}}{2}, cc_{top} + \frac{d_{i_{topo}}}{2}\right)}{d_{i_{topo}}}, 2.5\right)$	$c = 2.5$	
Tension development length past critical section:	$l_{d_{topo}} := \frac{3 \cdot f_{yT} \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \cdot \sqrt{f_c \cdot \text{psi}}} \cdot d_{i_{topo}}$	$l_{d_{topo}} = 25 \cdot \text{in}$	



## I. Calculate Actual Bottom Moment Capacity

Width of footing at section:

$$W_{bot\ q} := \text{if} \left( q \leq n, B + 2 \cdot x_{q}, D \right)$$

Number of bars within middle section:

$$n_{botm\ q} := \text{if} \left( W_m < \frac{W_{bot\ q}}{2}, \text{trunc} \left( \frac{W_m}{s_{botm}} \right), \text{trunc} \left( \frac{\frac{W_{bot\ q}}{2} - \sqrt{2} \cdot cc_{top}}{s_{botm}} \right) \right)$$

Spacing of first bar beyond the middle/outside boundary line:

$$s_{b1bar\ q} := \text{if} \left( W_m < \frac{W_{bot\ q}}{2}, s_{botm} \cdot n_{botm\ q} + s_{boto} - W_m, 0.0in \right)$$

Number of bars across bottom of footing at section:

$$n_{bot\ q} := \begin{cases} n_{botm\ q} + \left[ \text{trunc} \left[ \frac{0.5(W_{bot\ q} - 2W_m) - s_{b1bar\ q} - cc_{top}}{s_{boto}} \right] + 1 \right] & \text{if } W_m < \frac{W_{bot\ q}}{2} \\ n_{botm\ q} & \text{otherwise} \end{cases}$$

Bar counter:

$$ib := 1, 2 \dots n_{bot\ n+n_j}$$

Distance of bars from centerline across bottom of footing:

$$z_{bot,ib} := \text{if} \left[ ib \leq n_{botm\ n+n_j}, ib \cdot s_{botm}, s_{botm} \cdot n_{botm\ n+n_j} + s_{boto} \cdot (ib - n_{botm\ n+n_j}) \right]$$

Depth of footing for bottom middle steel at point:

$$d_{botm_i} := h_b + \frac{x_i}{a} \cdot h_c - cc_{bot} - di_{botm}$$

$$d_{botm_j} := h_b + h_c - cc_{bot} - di_{botm}$$

Depth of footing for bottom outside steel at point:

$$d_{boto_i} := h_b + \frac{x_i}{a} \cdot h_c - cc_{bot} - di_{boto}$$

$$d_{boto_j} := h_b + h_c - cc_{bot} - di_{boto}$$

Depth of each bar at section:

$$d_{barb_{q,ib}} := \begin{cases} \text{if } ib \cdot s_{botm} > \frac{B}{2} \\ \left| \begin{array}{l} d_{botm\ q} - \left( z_{bot,ib} - \frac{B}{2} \right) \cdot \frac{h_c}{a} \text{ if } ib \leq n_{botm\ q} \\ d_{boto\ q} - \left( z_{bot,ib} - \frac{B}{2} \right) \cdot \frac{h_c}{a} \text{ otherwise} \end{array} \right. \\ d_{botm\ q} \text{ otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{sbotm\ q} := \text{if} \left( x_q \geq cd_{bot}, n_{botm\ q} \cdot 2A_{botm}, \frac{1}{2} \cdot n_{botm\ q} \cdot 2A_{botm} \right) + A_{botm}$$

Area of steel provided across section at outside section:

$$A_{sboto\ q} := \text{if} \left[ x_q \geq cd_{bot}, (n_{bot\ q} - n_{botm\ q}) \cdot 2 \cdot A_{boto}, \frac{1}{2} \cdot (n_{bot\ q} - n_{botm\ q}) \cdot 2 \cdot A_{boto} \right]$$

Applying ACI minimum reinforcing requirements:

$$\rho_{B,min} := \max \left( 0.0014, \frac{0.0018 \cdot 60000 \cdot \text{psi}}{f_{yB}} \right) \quad \rho_{B,min} = 0.00144$$

Minimum area of steel  
required at section:

$$A_{sminb_q} := \rho_{B,min} \cdot \text{ConcreteVolume}(x_q) \quad (\text{Reference 1a})$$

Factored moment considering  
minimum reinforcing  
requirements at section:

$$M_{ubot_q} := \text{if}\left(A_{sbotm_q} + A_{sboto_q} \leq A_{sminb_q}, \text{"NG - BOTTOM R/F RATIO"}, \max(M_{ubot1_q}, 0)\right)$$

Footing is separated into strips  
containing one bar each. depth of  
compression block for each strip:

$$a_{botm} := \frac{A_{botm} \cdot f_{yB}}{0.85 \cdot f_c \cdot s_{botm}}$$

Depth of compression block for  
each outside strip:

$$a_{boto} := \frac{A_{boto} \cdot f_{yB}}{0.85 \cdot f_c \cdot s_{boto}}$$

Distance from section to end of bar  
for continuous bars:

$$l_{bot1_{q,ib}} := \text{if}\left[z_{bot_{ib}} > \frac{B}{2}, \max\left[x_q - \left(z_{bot_{ib}} - \frac{B}{2}\right) - \sqrt{2} \cdot cc_{top}, 0\right], \max(x_q - cc_{top}, 0)\right]$$

Distance from section to end of bar  
for cutoff bars:

$$l_{bot2_{q,ib}} := \max(x_q - cd_{bot}, 0)$$

Selection of appropriate bar end  
distance for bar in question:

$$l_{bot_{q,ib}} := \text{if}\left(ib \cdot s_{botm} - \frac{B}{2} < cd_{bot} \wedge \frac{ib}{2} \neq \text{trunc}\left(\frac{ib}{2}\right), l_{bot2_{q,ib}}, l_{bot1_{q,ib}}\right)$$

Factored moment capacity at section:

$$\phi M_{nbot_q} := \phi_b \cdot \left[ A_{botm} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,l}}}{l_{dbotm}}, 1\right) \cdot \left(d_{botm_q} - \frac{a_{botm}}{2}\right) + 2 \cdot \sum_{kk=1}^{n_{botm_q}} \left[ A_{botm} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,kk}}}{l_{dbotm}}, 1\right) \cdot \left(d_{barb_{q,kk}} - \frac{a_{botm}}{2}\right) \right] \dots \right. \\ \left. + 2 \cdot \sum_{jj=(n_{botm_q})+1}^{n_{bot_q}} \left[ A_{boto} \cdot f_{yB} \cdot \min\left(\frac{l_{bot_{q,jj}}}{l_{dboto}}, 1\right) \cdot \left(d_{barb_{q,jj}} - \frac{a_{boto}}{2}\right) \right] \right]$$

Check of factored moment vs.  
moment capacity at each section:

$$\text{check}_{bot_q} := \frac{M_{ubot_q}}{\phi M_{nbot_q}}$$

## J. Calculate Actual Top Moment Capacity

Applying ACI minimum reinforcing requirements:

$$\rho_{T,\min} := \max\left(\frac{3 \cdot \text{psi}^{0.5} \cdot \sqrt{f_c}}{f_{yT}}, \frac{200 \cdot \text{psi}}{f_{yT}}\right) \quad \rho_{T,\min} = 0.0028$$

Width of footing at section:

$$W_{\text{top}_q} := \text{if}(q \leq n, B + 2 \cdot x_{\text{alt}_q}, D)$$

Number of bars within middle section:

$$n_{\text{topm}_q} := \text{if}\left(W_m < \frac{W_{\text{top}_q}}{2}, \text{trunc}\left(\frac{W_m}{s_{\text{topm}}}\right), \text{trunc}\left(\frac{\frac{W_{\text{top}_q}}{2} - \sqrt{2} \cdot \text{cc}_{\text{top}}}{s_{\text{topm}}}\right)\right)$$

Spacing of first bar beyond the middle/outside boundary line:

$$s_{\text{t1bar}_q} := \text{if}\left(W_m < \frac{W_{\text{top}_q}}{2}, s_{\text{topm}} \cdot n_{\text{topm}_q} + s_{\text{topo}} - W_m, 0.0\text{in}\right)$$

Number of bars across top of footing at section:

$$n_{\text{top}_q} := \begin{cases} n_{\text{topm}_q} + \left\lceil \frac{0.5(W_{\text{top}_q} - 2W_m) - s_{\text{t1bar}_q} - \text{cc}_{\text{top}}}{s_{\text{topo}}} \right\rceil + 1 & \text{if } W_m < \frac{W_{\text{top}_q}}{2} \\ n_{\text{topm}_q} & \text{otherwise} \end{cases}$$

Bar counter:

$$it := 1, 2 \dots n_{\text{top}_{n+n_j}}$$

Distance of bars from centerline across top of footing:

$$z_{\text{top}_{it}} := \text{if}\left[it \leq n_{\text{top}_{n+n_j}}, it \cdot s_{\text{topm}}, s_{\text{topm}} \cdot n_{\text{top}_{n+n_j}} + s_{\text{topo}} \cdot \left(it - n_{\text{top}_{n+n_j}}\right)\right]$$

Depth of footing for top middle steel at point:

$$d_{\text{topm}_i} := h_b + \frac{x_{\text{alt}_i}}{a} \cdot h_c - \text{cc}_{\text{top}} - d_{i_{\text{topm}}}$$

$$d_{\text{topm}_j} := h_b + h_c - \text{cc}_{\text{top}} - d_{i_{\text{topm}}}$$

Depth of footing for top outside steel at point:

$$d_{\text{topo}_i} := h_b + \frac{x_{\text{alt}_i}}{a} \cdot h_c - \text{cc}_{\text{top}} - d_{i_{\text{topo}}}$$

$$d_{\text{topo}_j} := h_b + h_c - \text{cc}_{\text{top}} - d_{i_{\text{topo}}}$$

Depth of each bar at section:

$$d_{\text{bart}_{q,it}} := \begin{cases} \text{if } it \cdot s_{\text{topm}} > \frac{B}{2} \\ \left| \begin{array}{l} d_{\text{topm}_q} - \left(z_{\text{top}_{it}} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{if } it \leq n_{\text{topm}_q} \\ d_{\text{topo}_q} - \left(z_{\text{top}_{it}} - \frac{B}{2}\right) \cdot \frac{h_c}{a} \quad \text{otherwise} \end{array} \right| \\ d_{\text{topm}_q} \quad \text{otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{\text{stoptm}_q} := \text{if}\left(x_{\text{alt}_q} \geq \text{cd}_{\text{top}}, n_{\text{topm}_q} \cdot 2A_{\text{topm}}, \frac{1}{2} \cdot n_{\text{topm}_q} \cdot 2A_{\text{topm}}\right) + A_{\text{topm}}$$

Area of steel provided across section at outside section:

$$A_{\text{stopto}_q} := \text{if}\left[x_{\text{alt}_q} \geq \text{cd}_{\text{top}}, \left(n_{\text{top}_q} - n_{\text{topm}_q}\right) \cdot 2 \cdot A_{\text{topo}}, \frac{1}{2} \cdot \left(n_{\text{top}_q} - n_{\text{topm}_q}\right) \cdot 2 \cdot A_{\text{topo}}\right]$$

Minimum area of steel required at section:

$$A_{\text{smint}_q} := \rho_{T,\min} \cdot \text{ConcreteVolume}(x_q) \quad (\text{Reference 1a})$$

Factored moment considering minimum reinforcing requirements at section:

$$M_{utop\ q} := \text{if} \left( A_{stoptm\ q} + A_{stopo\ q} \leq A_{smint\ q}, \max \left( \frac{4}{3} \cdot M_{utop1\ q}, 0 \right), \max \left( M_{utop1\ q}, 0 \right) \right)$$

Distance from section to end of bar for continuous bars:

$$l_{top1\ q, it} := \text{if} \left[ z_{top, it} > \frac{B}{2}, \max \left[ x_{alt\ q} - \left( z_{top, it} - \frac{B}{2} \right) - \sqrt{2} \cdot cc_{top}, 0 \right], \max \left( x_{alt\ q} - cc_{top}, 0 \right) \right]$$

Distance from section to end of bar of bar for cut off bars:

$$l_{top2\ q, it} := \max \left( x_{alt\ q} - cd_{top}, 0 \right)$$

Selection of appropriate bar end distance for bar in question:

$$l_{top\ q, it} := \text{if} \left( it \cdot s_{topm} - \frac{B}{2} < cd_{top} \wedge \frac{it}{2} \neq \text{trunc} \left( \frac{it}{2} \right), l_{top2\ q, it}, l_{top1\ q, it} \right)$$

Force developed in each middle bar including development length of individual bars:

$$f_{sm\ q, it} := \text{if} \left( it \leq n_{top\ q}, A_{topm} \cdot f_{yT} \cdot \min \left( \frac{l_{top\ q, it}}{l_{dtopm}}, 1 \right), 0 \right)$$

Force developed in each outside bar including development length of individual bars:

$$f_{so\ q, it} := \text{if} \left( it \leq n_{top\ q}, A_{topo} \cdot f_{yT} \cdot \min \left( \frac{l_{top\ q, it}}{l_{dtopo}}, 1 \right), 0 \right)$$

Beta factor:

$$\beta_1 := \text{if} \left[ f_c \geq 4000 \text{psi}, \max \left[ 0.85 - 0.05 \cdot \left( \frac{\frac{f_c}{\text{psi}} - 4000}{1000} \right), 0.65 \right], 0.85 \right] \beta_1 = 0.80$$

Depth of neutral axis at section:

$$xop\ q := \frac{A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt\ q} - cc_{top}}{l_{dtopm}}, 1 \right) + 2 \cdot \sum_{kk=1}^{n_{topm\ q}} f_{sm\ q, kk} + 2 \cdot \sum_{jj=(n_{topm\ q})+1}^{n_{top\ q}} f_{so\ q, jj}}{W_{top\ q} \cdot \beta_1 \cdot 0.85 \cdot f_c}$$

Factored moment capacity at section:

$$\phi M_{ntop\ q} := \phi_b \cdot \left[ A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt\ q} - cc_{top}}{l_{dtopm}}, 1 \right) \cdot (d_{bart\ q, 1} - xop\ q) + W_{top\ q} \cdot 0.85 \cdot \beta_1 \cdot f_c \cdot xop\ q \cdot \left[ xop\ q - \left( \frac{xop\ q \cdot \beta_1}{2} \right) \right] \dots \right. \\ \left. + 2 \cdot \sum_{kk=1}^{n_{topm\ q}} [f_{sm\ q, kk} \cdot (d_{bart\ q, kk} - xop\ q)] + 2 \cdot \sum_{jj=(n_{topm\ q})+1}^{n_{top\ q}} [f_{so\ q, jj} \cdot (d_{bart\ q, jj} - xop\ q)] \right]$$

Check of factored moment vs. moment capacity at each section:

$$\text{check}_{top\ q} := \frac{M_{utop\ q}}{\phi M_{ntop\ q}}$$

Factored moment capacity in middle section at critical section:

$$\phi M_{nbotm_{n+n_j}} := \phi_b \cdot \left[ A_{botm} \cdot f_{yB} \cdot \min \left[ \frac{I_{bot(n+n_j),1}}{I_{dbotm}}, 1 \right] \cdot \left( d_{botm_{n+n_j}} - \frac{a_{botm}}{2} \right) + 2 \cdot \sum_{kk=1}^{n_{botm_{n+n_j}}} \left[ A_{botm} \cdot f_{yB} \cdot \min \left[ \frac{I_{bot(n+n_j),kk}}{I_{dbotm}}, 1 \right] \cdot \left[ d_{barb_{(n+n_j),kk}} - \frac{a_{botm}}{2} \right] \right] \right]$$

$$\phi M_{nbotm_{n+n_j}} = 20250 \cdot \text{kip} \cdot \text{ft}$$

Factored moment capacity in middle section at critical section:

$$\phi M_{ntopm_{n+n_j}} := \phi_b \cdot \left[ A_{topm} \cdot f_{yT} \cdot \min \left( \frac{x_{alt_{n+n_j}} - cc_{top}}{I_{dtopm}}, 1 \right) \cdot \left( d_{bart_{n+n_j,1}} - x_{op_{n+n_j}} \right) + W_m \cdot 0.85 \cdot \beta_1 \cdot f_c \cdot x_{op_{n+n_j}} \cdot \left[ x_{op_{n+n_j}} - \left( \frac{x_{op_{n+n_j}} \cdot \beta_1}{2} \right) \right] \dots \right]$$

$$+ 2 \cdot \sum_{kk=1}^{n_{topm_{n+n_j}}} \left[ f_{sm_{n+n_j,kk}} \cdot \left( d_{bart_{n+n_j,kk}} - x_{op_{n+n_j}} \right) \right]$$

$$\phi M_{ntopm_{n+n_j}} = 6774 \cdot \text{kip} \cdot \text{ft}$$

Unbalanced wind moment on joint:

$$M_{unbalancedW} := M_{dW}$$

$$M_{unbalancedW} = 31968 \cdot \text{kip} \cdot \text{ft}$$

Fraction of wind moment carried by flexure:

$$\gamma_{fW} := \frac{\phi M_{nbotm_{n+n_j}} + \phi M_{ntopm_{n+n_j}}}{M_{unbalancedW}}$$

$$\gamma_{fW} = 0.85$$

Unbalanced seismic moment on joint:

$$M_{unbalancedEQ} := M_{dEQ}$$

$$M_{unbalancedEQ} = 15510 \cdot \text{kip} \cdot \text{ft}$$

Fraction of seismic moment carried by flexure:

$$\gamma_{fEQ} := \frac{\phi M_{nbotm_{n+n_j}} + \phi M_{ntopm_{n+n_j}}}{M_{unbalancedEQ}}$$

$$\gamma_{fEQ} = 1.74$$

### K. Bottom Moment Capacity Results

$q =$	$x_q =$	$W_{bot\ q} =$	$A_{sboto\ q} + A_{sbotm\ q} =$	$A_{sminb\ q} =$	$M_{ubot\ q} =$	$\phi M_{nbot\ q} =$	$check_{bot\ q} =$
	ft	ft	$\cdot in^2$	$\cdot in^2$	$\cdot k \cdot ft$	$\cdot k \cdot ft$	
1	0.5	21.7	45.2	10.9	0	648	0.00
2	1.0	22.7	45.2	11.9	0	1546	0.00
3	1.5	23.7	48.4	12.8	0	2507	0.00
4	2.0	24.7	51.5	13.8	0	3581	0.00
5	2.5	25.7	51.5	14.8	52	4838	0.01
6	3.0	26.7	54.6	15.8	142	6170	0.02
7	3.5	27.7	57.7	16.8	265	7625	0.03
8	4.0	28.7	57.7	17.9	422	8903	0.05
9	4.5	29.7	60.8	18.9	612	9537	0.06
10	5.0	30.7	64.0	20.0	834	10188	0.08
11	5.5	31.7	64.0	21.1	1090	10908	0.10
12	6.0	32.7	68.3	22.3	1378	11556	0.12
13	6.5	33.7	68.3	23.4	1699	12262	0.14
14	7.0	34.7	68.3	24.6	2052	12967	0.16
15	7.5	35.7	69.5	25.8	2437	13665	0.18
16	8.0	36.7	70.7	27.0	2858	14368	0.20
17	8.5	37.7	70.7	28.2	3307	15064	0.22
18	9.0	38.7	71.9	29.5	3789	15725	0.24
19	9.5	39.7	73.1	30.7	4302	16360	0.26
20	10.0	40.7	73.1	32.0	4846	16942	0.29
21	10.5	41.7	74.3	33.3	5421	17534	0.31
22	11.0	42.7	75.5	34.7	6028	18133	0.33
23	11.5	43.7	75.5	36.0	6666	18730	0.36
24	12.0	44.7	76.7	37.4	7334	19337	0.38
25	12.5	45.7	77.9	38.8	8033	19952	0.40
26	13.0	46.7	77.9	40.2	8762	20565	0.43
27	13.5	47.7	79.1	41.6	9507	21188	0.45
28	14.0	48.7	80.3	43.1	10244	21818	0.47
29	14.5	49.7	80.3	44.5	10974	22446	0.49
30	15.1	50.0	81.5	45.0	11830	22737	0.52
31	15.7	50.0	81.5	45.0	12674	22852	0.55
32	16.3	50.0	81.5	45.0	13507	22932	0.59
33	16.9	50.0	81.5	45.0	14329	22974	0.62
34	17.5	50.0	81.5	45.0	15139	22979	0.66

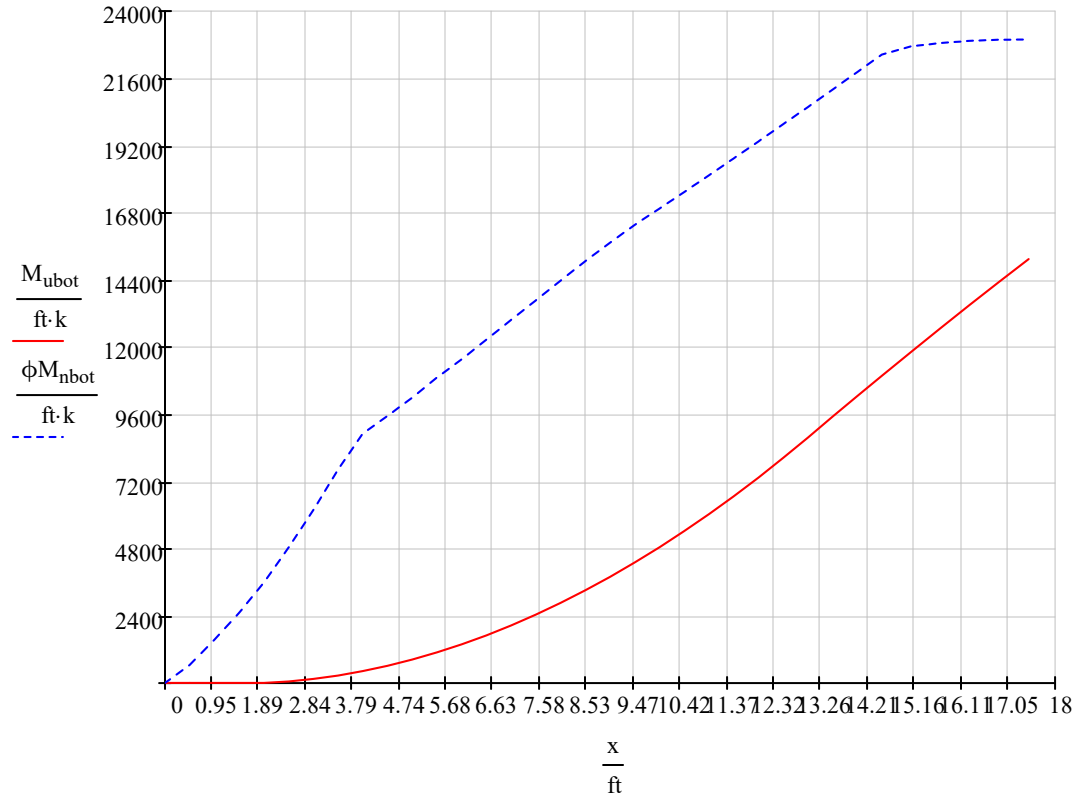
$check_{bot\ n+n_j} = 0.66$

**L. Top Moment Capacity Results**

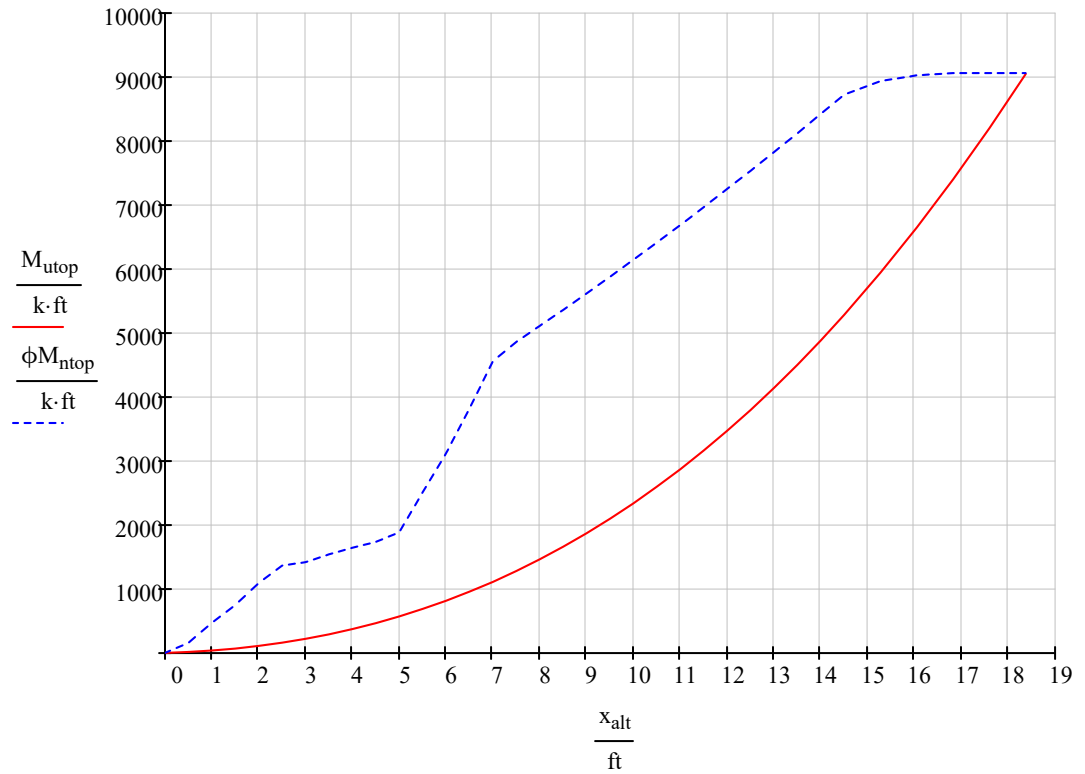
q =	$x_{alt\ q} =$	$W_{top\ q} =$	$A_{stopo\ q} + A_{stopm\ q} =$	$A_{smint\ q} =$	$M_{utop\ q} =$	$\phi M_{ntop\ q} =$	check <sub>top<sub>q</sub></sub> =
		ft	$\cdot in^2$	$\cdot in^2$	$\cdot ft \cdot k$	$\cdot ft \cdot k$	
1	0.50	21.7	7.0	21.5	14	158	0.09
2	1.00	22.7	7.5	23.3	37	469	0.08
3	1.50	23.7	7.9	25.2	69	748	0.09
4	2.00	24.7	8.4	27.1	109	1093	0.10
5	2.50	25.7	8.4	29.0	160	1365	0.12
6	3.00	26.7	8.8	31.0	220	1418	0.16
7	3.50	27.7	9.2	33.0	291	1541	0.19
8	4.00	28.7	9.7	35.1	373	1643	0.23
9	4.50	29.7	9.7	37.2	466	1734	0.27
10	5.00	30.7	19.8	39.3	570	1885	0.30
11	5.50	31.7	20.7	41.5	686	2508	0.27
12	6.00	32.7	22.4	43.7	815	3119	0.26
13	6.50	33.7	22.4	46.0	957	3821	0.25
14	7.00	34.7	22.4	48.3	1111	4560	0.24
15	7.50	35.7	23.3	50.6	1279	4861	0.26
16	8.00	36.7	24.2	53.0	1462	5110	0.29
17	8.50	37.7	25.1	55.4	1659	5359	0.31
18	9.00	38.7	25.1	57.9	1870	5616	0.33
19	9.50	39.7	26.0	60.4	2097	5878	0.36
20	10.00	40.7	26.8	62.9	2339	6145	0.38
21	10.50	41.7	27.7	65.5	2596	6411	0.40
22	11.00	42.7	27.7	68.1	2871	6686	0.43
23	11.50	43.7	28.6	70.7	3162	6965	0.45
24	12.00	44.7	29.5	73.4	3470	7249	0.48
25	12.50	45.7	30.4	76.2	3796	7533	0.50
26	13.00	46.7	30.4	78.9	4139	7825	0.53
27	13.50	47.7	31.2	81.7	4501	8121	0.55
28	14.00	48.7	32.1	84.6	4882	8422	0.58
29	14.50	49.7	33.0	87.5	5282	8723	0.61
30	15.28	50.0	33.0	88.3	5941	8936	0.66
31	16.05	50.0	33.0	88.3	6647	9026	0.74
32	16.83	50.0	33.0	88.3	7400	9060	0.82
33	17.60	50.0	33.0	88.3	8200	9060	0.91
34	18.38	50.0	33.0	88.3	9047	9060	1.00

check<sub>top<sub>n+n<sub>j</sub></sub></sub> = 1.00

**Factored Moment vs.  
 Moment Capacity -  
 Bottom Reinforcing**



**Factored Moment vs.  
 Moment Capacity -  
 Top Reinforcing**





### M. Check Cutoff Locations

(Reference 1a)

Distance to top bar cutoff location from edge of footing:  $cd_{top} = 5.0 \cdot ft$

Counter corresponding to cutoff location:  $b := \frac{cd_{top}}{x_{alt_1}}$

Distance to cutoff location from edge of footing:  $x_{alt_b} = 5.0 \cdot ft$

Factored moment capacity at cutoff location:  $\phi M_{ntop_b} = 1885 \cdot k \cdot ft$

Footing effective depth at cutoff location:  $d_{topm_b} = 3.1 \cdot ft$

Distance equal to effective depth from cutoff location:  $cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm}) = 8.1 \cdot ft$

Counter corresponding to bin below distance equal to effective depth from cutoff location:  $b_{under} := \text{trunc}\left(2 \cdot \frac{cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm})}{ft}\right) \quad b_{under} = 16$

$x_{alt_{b_{under}}} = 8.0 \cdot ft$

Moment at distance equal to effective depth from cut:

$$M_{utopcutd} := M_{utop_{b_{under}}} \dots$$

$$+ \left( cd_{top} + \max(d_{topm_b}, 12 \cdot di_{topm}) - x_{alt_{b_{under}}} \right) \cdot \left[ \frac{M_{utop_{(b_{under}+1)}} - M_{utop_{b_{under}}}}{0.5ft} \right]$$

$$M_{utopcutd} = 1490 \cdot k \cdot ft$$

Check factored moment at distance equal to effective depth from cutoff location:

$$\frac{M_{utopcutd}}{\phi M_{ntop_b}} = 0.79$$

Distance to bottom bar cutoff location from edge of footing:

$$cd_{bot} = 0.0 \text{ ft}$$

Counter corresponding to cutoff location:

$$b := \frac{cd_{bot}}{x_1}$$

Distance to cutoff location from edge of footing:

$$x_b = 0.0 \text{ ft}$$

Factored moment capacity at cutoff location:

$$\phi M_{nbot_b} = 0 \cdot k \cdot ft$$

Footing effective depth at cutoff location:

$$d_{botm_b} = 0.0 \text{ ft}$$

Distance equal to effective depth from cutoff location:

$$cd_{bot} + \max(d_{botm_b}, di_{botm}) = 0.1 \text{ ft}$$

Counter corresponding to bin below distance equal to effective depth from cutoff location:

$$b_{under2} := \text{trunc} \left( 2 \cdot \frac{cd_{bot} + \max(d_{botm_b}, di_{botm})}{ft} \right) \quad b_{under2} = 0$$

$$x_{b_{under2}} = 0.0 \text{ ft}$$

Moment at distance equal to effective depth from cut:

$$M_{ubotcutd} := M_{ubot_{b_{under2}}} \dots$$

$$+ \left( cd_{bot} + \max(d_{botm_b}, di_{botm}) - x_{b_{under2}} \right) \cdot \left[ \frac{M_{ubot_{(b_{under2}+1)}} - M_{ubot_{b_{under2}}}}{0.5ft} \right]$$

$$M_{ubotcutd} = 0 \cdot k \cdot ft$$

Check factored moment at distance equal to effective depth from cutoff location:

$$\frac{M_{ubotcutd}}{\phi M_{nbot_b}} = 0.00$$

## IX-b. Moment Capacity of Bottom Reinforcement at 45 degree angle

Distance to critical section from centerline of foundation:

$$x_o := \frac{\sqrt{\pi} \cdot C}{4} \quad x_o = 90.4 \cdot \text{in}$$

Slant distance on critical section from foundation edge to slope transition point:

$$a_p := \left(\frac{1}{2}\right) \cdot \sqrt{2 \cdot \left(\frac{D - B}{2}\right)^2} - x_o \quad a_p = 33.9 \cdot \text{in}$$

Geometric distance to transition point:

$$C_1 := \sqrt{2} \cdot a_p \quad C_1 = 47.9 \cdot \text{in}$$

Height of transition point:

$$h := h_b + \left(\frac{h_c}{a}\right) \cdot C_1 \quad h = 37.3 \cdot \text{in}$$

Number of bars between centerline and critical section:

$$N_{\text{barsdown}} := \text{trunc}\left(\frac{x_o}{\sqrt{2} \cdot s_{\text{botm}}}\right) \quad N_{\text{barsdown}} = 7$$

Spacing to first bar on critical section:

$$x_1 := \sqrt{2} \cdot s_{\text{botm}} - (x_o - \sqrt{2} \cdot s_{\text{botm}} \cdot N_{\text{barsdown}}) \quad x_1 = 11.43 \cdot \text{in}$$

Number of middle section bars:

$$N_{\text{barbm}} := n_{\text{botm}_{n+n_j}} - N_{\text{barsdown}} \quad N_{\text{barbm}} = 14$$

Distance to last middle bar:

$$d_{\text{tp}} := \sqrt{2} \cdot (N_{\text{barbm}} - 1) \cdot s_{\text{botm}} + x_1 \quad d_{\text{tp}} = 177 \cdot \text{in}$$

Number of outside section bars:

$$N_{\text{barbo}} := \text{trunc}\left(\frac{\frac{D}{2} - cc_{\text{top}} - d_{\text{tp}}}{\sqrt{2} \cdot s_{\text{boto}}}\right) \quad N_{\text{barbo}} = 9$$

Number of bars crossing the critical section in diagonal direction 1:

$$N_{\text{bars1}} := N_{\text{barbm}} + N_{\text{barbo}} \quad N_{\text{bars1}} = 23$$

Bar counter for diagonal bars in diagonal direction 1:

$$b_1 := 1, 2 \dots N_{\text{bars1}}$$

Distance from centerline of bars in diagonal direction 1:

$$x_{b_1} := \text{if}\left[b_1 > N_{\text{barbm}}, x_{N_{\text{barbm}}} + \sqrt{2} \cdot s_{\text{boto}} \cdot (b_1 - N_{\text{barbm}}), x_1 + \sqrt{2} \cdot s_{\text{botm}} \cdot (b_1 - 1)\right]$$

Development lengths provided for individual bars:

$$L_{\text{in}_{b_1}} := \min\left[100 \cdot \text{in}, \sqrt{2} \cdot \left(\frac{D}{2} - cc_{\text{top}} - x_{b_1}\right)\right] \quad L_{\text{out}_{b_1}} := 100 \cdot \text{in}$$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{\text{bot}} - di_{\text{botm}} & \text{if } x_{b_1} \leq \frac{D}{2} - (a_p + 2 \cdot x_o) \\ \frac{h_b - h}{a_p} \cdot \left[x_{b_1} - \left(\frac{D}{2} - a_p\right)\right] + h - (cc_{\text{bot}} + di_{\text{botm}}) & \text{if } x_{b_1} > \frac{D}{2} - a_p \\ \frac{h - (h_b + h_c)}{2 \cdot x_o} \cdot \left[x_{b_1} - \left[\frac{D}{2} - (a_p + 2 \cdot x_o)\right]\right] + h_c + h_b - (cc_{\text{bot}} + di_{\text{botm}}) & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \text{trunc} \left[ \frac{\left( \frac{D}{2} - cc_{top} \right) - \left( \sqrt{2} \cdot s_{botm} - x_1 \right)}{\sqrt{2} \cdot s_{botm}} + 1 \right] \quad N_{bars2} = 24$$

Average width of tributary area for individual bars:

$$b := \frac{\frac{D}{2}}{N_{bars1} + N_{bars2}} \quad b = 6.38 \cdot \text{in}$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma_{1_{b1}} := \begin{cases} \frac{\sqrt{2} \cdot f_{yB} \cdot A_{boto}}{2} \cdot \min \left( \frac{\min(L_{in_{b1}}, L_{out_{b1}})}{l_{dboto}}, 1 \right) & \text{if } b_1 > N_{barbm} \\ \frac{\sqrt{2} \cdot f_{yB} \cdot A_{botm}}{2} \cdot \min \left( \frac{\min(L_{in_{b1}}, L_{out_{b1}})}{l_{dbotm}}, 1 \right) & \text{otherwise} \end{cases}$$

Depth of compression block for bars in diagonal direction 1:

$$a_{1_{b1}} := \frac{\sigma_{1_{b1}}}{0.85 \cdot f_c \cdot b}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 \dots N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2} \cdot s_{botm} \cdot (b_2) - 2 \cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$\text{Centerbar} := N_{barsdown} + 1 \quad \text{Centerbar} = 8$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \frac{\text{Centerbar}}{2} > \text{trunc} \left( \frac{\text{Centerbar}}{2} \right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$\text{Other}_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \quad \text{Other}_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} \text{Other}_{bars} & \text{if } \frac{b_2}{2} > \text{trunc} \left( \frac{b_2}{2} \right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b_2}} := 100 \text{in}$$

$L_{out2_{b_2}} :=$  if  $C_{bar_{b_2}} = \text{"noncutoff"}$

$$\begin{cases} \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left( \frac{D}{2} - cc_{top} - \lambda_{b_2} \right) \right] & \text{if } \frac{D}{2} - \lambda_{b_2} < \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \\ \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b_2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] \right] \right] & \text{otherwise} \\ \max \left[ 0 \text{in}, \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b_2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_o \right] \right] - (cd_{bot} - cc_{top}) \right] \right] & \text{otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b2} := \begin{cases} h_b + h_c - cc_{bot} - di_{botm} & \text{if } \lambda_{b2} \leq \frac{D}{2} - (a_p + 2 \cdot x_o) \\ \frac{h_b - h}{a_p} \cdot \left[ \lambda_{b2} - \left( \frac{D}{2} - a_p \right) \right] + h - (cc_{bot} + di_{botm}) & \text{if } \lambda_{b2} > \frac{D}{2} - a_p \\ \frac{h - (h_b + h_c)}{2 \cdot x_o} \cdot \left[ \lambda_{b2} - \left[ \frac{D}{2} - (a_p + 2 \cdot x_o) \right] \right] + h_c + h_b - (cc_{bot} + di_{botm}) & \text{otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma2_{b2} := \frac{\sqrt{2} \cdot f_{yB} \cdot A_{botm}}{2} \cdot \min \left( \frac{\min(L_{in2_{b2}}, L_{out2_{b2}})}{l_{dbotm}}, 1 \right)$$

Depth of compression block for bars in diagonal direction 2:

$$a2_{b2} := \frac{\sigma2_{b2}}{0.85 \cdot f_c \cdot b}$$

Factored moment capacity at section:

$$\phi M_{nbot} := \phi_b \cdot \left[ 2 \cdot \left[ \sum_{kk=1}^{N_{bars1}} \left[ \sigma1_{kk} \cdot \left( d_{kk} - \frac{a1_{kk}}{2} \right) \right] \right] + 2 \cdot \left[ \sum_{kk=1}^{N_{bars2}} \left[ \sigma2_{kk} \cdot \left( d2_{kk} - \frac{a2_{kk}}{2} \right) \right] \right] \right] \dots \phi M_{nbot} = 23730 \cdot k \cdot ft$$

Ultimate moment in bottom reinforcement at critical section:

$$M_{ubot_{n+nj}} = 15139 \cdot k \cdot ft$$

Check of factored moment against moment capacity at critical section:

$$check_{bot} := \frac{M_{ubot_{n+nj}}}{\phi M_{nbot}} \quad \boxed{check_{bot} = 0.64}$$

### IX-c. Moment Capacity of Top Reinforcement at 45 degree angle

Distance to alternate critical section from centerline of foundation:

$$x_{alt.o} := \frac{\sqrt{\pi} \cdot OD}{4} \quad x_{alt.o} = 79.5 \cdot \text{in}$$

Slant distance on alternate critical section from foundation edge to slope transition point:

$$a_{alt.p} := \left(\frac{1}{2}\right) \cdot \sqrt{2 \cdot \left(\frac{D-B}{2}\right)^2} - x_{alt.o} \quad a_{alt.p} = 44.8 \cdot \text{in}$$

Geometric distance to transition point:

$$C_{alt.1} := \sqrt{2} \cdot a_{alt.p} \quad C_{alt.1} = 63.3 \cdot \text{in}$$

Height of transition point:

$$h_{alt} := h_b + \left(\frac{h_c}{a}\right) \cdot C_{alt.1} \quad h_{alt} = 40.3 \cdot \text{in}$$

Vertical distance to the centerline of top bars in outer section:

$$K_1 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h_{alt} - h_b}{a_{alt.p}}\right)\right)} \quad K_1 = 2.86 \cdot \text{in}$$

Vertical distance to the centerline of top bars in middle section:

$$K_2 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h_b + h_c - h_{alt}}{2 \cdot x_{alt.o}}\right)\right)} \quad K_2 = 2.78 \cdot \text{in}$$

Number of bars between centerline and critical section:

$$N_{barsdown} := \text{trunc}\left(\frac{x_{alt.o}}{\sqrt{2} \cdot s_{topm}}\right) \quad N_{barsdown} = 7$$

Spacing to first bar on critical section:

$$x_1 := \sqrt{2} \cdot s_{topm} - (x_{alt.o} - \sqrt{2} \cdot s_{topm} \cdot N_{barsdown}) \quad x_1 = 11.03 \cdot \text{in}$$

Number of middle section bars:

$$N_{bartm} := n_{topm_{n+n_j}} - N_{barsdown} \quad N_{bartm} = 17$$

Distance to last middle bar:

$$d_{tp} := \sqrt{2} \cdot (N_{bartm} - 1) \cdot s_{topm} + x_1 \quad d_{tp} = 192 \cdot \text{in}$$

Number of outside section bars:

$$N_{barto} := \text{trunc}\left(\frac{\left(\frac{D}{2} - cc_{top} - d_{tp}\right)}{\sqrt{2} \cdot s_{topo}}\right) \quad N_{barto} = 9$$

Number of bars crossing the critical section in diagonal direction 1:

$$N_{bars1} := N_{bartm} + N_{barto} \quad N_{bars1} = 26$$

Bar counter for diagonal bars in diagonal direction 1:

$$b_1 := 1, 2 \dots N_{bars1}$$

Distance from centerline of bars in diagonal direction 1:

$$x_{b_1} := \text{if}\left[b_1 > N_{bartm}, x_{N_{bartm}} + \sqrt{2} \cdot s_{topo} \cdot (b_1 - N_{bartm}), x_1 + \sqrt{2} \cdot s_{topm} \cdot (b_1 - 1)\right]$$

Development lengths provided for individual bars:

$$L_{in_{b_1}} := \min\left[100 \cdot \text{in}, \sqrt{2} \cdot \left(\frac{D}{2} - cc_{top} - x_{b_1}\right)\right] \quad L_{out_{b_1}} := 100 \cdot \text{in}$$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{top} - di_{topm} & \text{if } x_{b_1} \leq \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \\ \frac{h_b - h_{alt}}{a_{alt.p}} \cdot \left[ x_{b_1} - \left( \frac{D}{2} - a_{alt.p} \right) \right] + h_{alt} - K_1 & \text{if } x_{b_1} > \frac{D}{2} - a_{alt.p} \\ \frac{h_{alt} - (h_b + h_c)}{2 \cdot x_{alt.o}} \cdot \left[ x_{b_1} - \left[ \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \right] \right] + h_c + h_b - K_2 & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \text{trunc} \left[ \frac{\left( \frac{D}{2} - cc_{top} \right) - \left( \sqrt{2} \cdot s_{topm} - x_1 \right)}{\sqrt{2} \cdot s_{topm}} + 1 \right] \quad N_{bars2} = 27$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma_{l_{b_1}} := \begin{cases} \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topo}}{2} \cdot \min \left( \frac{\min(L_{in_{b_1}}, L_{out_{b_1}})}{l_{dtopo}}, 1 \right) & \text{if } b_1 > N_{bartm} \\ \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topm}}{2} \cdot \min \left( \frac{\min(L_{in_{b_1}}, L_{out_{b_1}})}{l_{dtopm}}, 1 \right) & \text{otherwise} \end{cases}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 \dots N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2} \cdot s_{topm} \cdot (b_2) - 2 \cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$\text{Centerbar} := N_{barsdown} + 1$$

$$\text{Centerbar} = 8$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \frac{\text{Centerbar}}{2} > \text{trunc} \left( \frac{\text{Centerbar}}{2} \right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$\text{Other}_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \quad \text{Other}_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} \text{Other}_{bars} & \text{if } \frac{b_2}{2} > \text{trunc} \left( \frac{b_2}{2} \right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b2}} := 100\text{in}$$

$$L_{out2_{b2}} := \begin{cases} \text{if } C_{bar_{b2}} = \text{"noncutoff"} \\ \left[ \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left( \frac{D}{2} - cc_{top} - \lambda_{b2} \right) \right] \text{ if } \frac{D}{2} - \lambda_{b2} < \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right. \\ \left. \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] \right] \right] \right] \text{ otherwise} \\ \max \left[ 0 \text{in}, \min \left[ 100 \cdot \text{in}, \sqrt{2} \cdot \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] - cc_{top} + \frac{\sqrt{2}}{2} \cdot \left[ \frac{D}{2} - \lambda_{b2} - \left[ \frac{\sqrt{2}}{4} \cdot (D - B) - x_{alt.o} \right] \right] - (cd_{top} - cc_{top}) \right] \right] \text{ otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b2} := \begin{cases} h_b + h_c - cc_{top} - d_{i_{topm}} \text{ if } \lambda_{b2} \leq \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \\ \frac{h_b - h_{alt}}{a_{alt.p}} \cdot \left[ \lambda_{b2} - \left( \frac{D}{2} - a_{alt.p} \right) \right] + h_{alt} - K_1 \text{ if } \lambda_{b2} > \frac{D}{2} - a_{alt.p} \\ \frac{h_{alt} - (h_b + h_c)}{2 \cdot x_{alt.o}} \cdot \left[ \lambda_{b2} - \left[ \frac{D}{2} - (a_{alt.p} + 2 \cdot x_{alt.o}) \right] \right] + h_c + h_b - K_2 \text{ otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma_{2_{b2}} := \frac{\sqrt{2} \cdot f_{yT} \cdot A_{topm}}{2} \cdot \min \left( \frac{\min(L_{in2_{b2}}, L_{out2_{b2}})}{l_{dtopm}}, 1 \right)$$

Footing is separated into strips containing one bar each. Depth of compression block for each strip:

$$a_{top} := \frac{\sum_{ii=1}^{N_{bars1}} \sigma_{1_{ii}} + \sum_{jj=1}^{N_{bars2}} \sigma_{2_{jj}}}{0.85 \cdot f_c \cdot \frac{D}{2}} \quad a_{top} = 0.90 \cdot \text{in}$$

Factored moment capacity at critical section:

$$\phi M_{ntop} := \phi_b \cdot \left[ 2 \cdot \left[ \sum_{kk=1}^{N_{bars1}} \left[ \sigma_{1_{kk}} \cdot \left( d_{kk} - \frac{a_{top}}{2} \right) \right] \right] \dots \right. \\ \left. + 2 \cdot \left[ \sum_{kk=1}^{N_{bars2}} \left[ \sigma_{2_{kk}} \cdot \left( d2_{kk} - \frac{a_{top}}{2} \right) \right] \right] \right] \quad \phi M_{ntop} = 8662 \cdot \text{k} \cdot \text{ft}$$

Ultimate moment in top reinforcement:

$$M_{utop_{n+nj}} = 9047 \cdot \text{k} \cdot \text{ft}$$

Check of factored moment against moment capacity at critical section:

$$check_{top} := \frac{M_{utop_{n+nj}}}{\phi M_{ntop}} \quad \boxed{check_{top} = 1.04}$$



### IX-d. One-Way Shear Capacity Check

Plan location of section:  $x_i := \frac{i}{2} \cdot \text{ft}$        $x_j := \max(x) + (x_{\text{face}} - \max(x)) \cdot \frac{j - n}{n_j}$

Depth, d, as a function of distance along the sloped portion of the foundation is:  $d_i := h_b + \frac{x_i}{a} \cdot h_c - cc_{\text{bot}} - \frac{d_{\text{botm}}}{2}$

$$d_j := h_b + h_c - cc_{\text{bot}} - \frac{d_{\text{botm}}}{2}$$

Determine controlling load case:  $V_{u_q} := \text{if} \left( |V_{uW_q}| \geq |V_{uEQ_q}|, V_{uW_q}, V_{uEQ_q} \right)$

Location of critical section from edge of footing:  $x_{\text{critical}} := x_{\text{face}} - d_{n+n_j}$        $x_{\text{critical}} = 12.6 \text{ ft}$  (Reference 1a)

Array counter for all slices up to the critical section:  $\text{qcs} := 1, 2 \dots \text{trunc} \left( \frac{x_{\text{critical}}}{0.5 \text{ ft}} \right) + 1$

Shear capacity between edge and a:  $\phi V_{n_i} := \phi_v \cdot 2 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[ B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right) \right]$  (Reference 1a)

Shear capacity between a and  $x_{\text{face}}$ :  $\phi V_{n_j} := \phi_v \cdot 2 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[ B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right) \right]$  (Reference 1a)

Reinforcing ratio at section between edge and a:  $\text{rho}_{x_i} := \frac{A_{\text{sboto}_i} + A_{\text{sbotm}_i}}{B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right)}$

Reinforcing ratio at section between a and edge:  $\text{rho}_{x_j} := \frac{A_{\text{sboto}_j} + A_{\text{sbotm}_j}}{B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right)}$

Capacity between edge and a:  $\phi V_{n2_i} := \phi_v \cdot \left( 8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \right) \cdot (\text{rho}_{x_i})^{\frac{1}{3}} \cdot \left[ B \cdot d_i + 2 \cdot x_i \cdot \left( d_i - x_i \cdot \frac{h_c}{2 \cdot a} \right) \right]$

Capacity between a and  $x_{\text{face}}$ :  $\phi V_{n2_j} := \phi_v \cdot \left( 8 \cdot \text{psi}^{\frac{1}{2}} \cdot \sqrt{f_c} \right) \cdot (\text{rho}_{x_j})^{\frac{1}{3}} \cdot \left[ B \cdot d_j + 2 \cdot a \cdot \left( d_j - \frac{h_c}{2} \right) \right]$

Shear Design Results

qcs =	$x_{qcs} =$ ft	$d_{qcs} =$ in	$V_{u_{qcs}} =$ ·k	$\phi V_{n_{qcs}} =$ ·k	$\phi V_{n2_{qcs}} =$ ·k	$\frac{V_{u_{qcs}}}{\phi V_{n_{qcs}}} =$	$\frac{V_{u_{qcs}}}{\phi V_{n2_{qcs}}} =$
1	0.5	25.5	-18	703	533	-0.03	-0.03
2	1.0	26.6	-25	766	565	-0.03	-0.04
3	1.5	27.8	-32	832	610	-0.04	-0.05
4	2.0	28.9	78	898	656	0.09	0.12
5	2.5	30.1	146	967	689	0.15	0.21
6	3.0	31.3	213	1036	736	0.21	0.29
7	3.5	32.4	280	1107	783	0.25	0.36
8	4.0	33.6	346	1180	817	0.29	0.42
9	4.5	34.7	412	1254	866	0.33	0.48
10	5.0	35.9	478	1330	916	0.36	0.52
11	5.5	37.1	544	1407	951	0.39	0.57
12	6.0	38.2	609	1485	1007	0.41	0.60
13	6.5	39.4	674	1565	1043	0.43	0.65
14	7.0	40.5	739	1647	1079	0.45	0.68
15	7.5	41.7	803	1729	1122	0.46	0.72
16	8.0	42.9	867	1814	1164	0.48	0.74
17	8.5	44.0	931	1900	1201	0.49	0.78
18	9.0	45.2	994	1987	1244	0.50	0.80
19	9.5	46.4	1057	2076	1288	0.51	0.82
20	10.0	47.5	1120	2166	1325	0.52	0.84
21	10.5	48.7	1182	2258	1370	0.52	0.86
22	11.0	49.8	1244	2351	1415	0.53	0.88
23	11.5	51.0	1306	2446	1453	0.53	0.90
24	12.0	52.2	1367	2542	1498	0.54	0.91
25	12.5	53.3	1428	2640	1545	0.54	0.92
26	13.0	54.5	1489	2739	1583	0.54	0.94

$$ii := 1 .. \text{trunc}\left(\frac{x_{\text{critical}}}{0.5\text{ft}}\right) + 1$$

$$\text{vals}_{ii} := \frac{V_{u_{ii}}}{\phi V_{n_{ii}}}$$

$$\text{Check}_{\text{Shear1}} := \max(\text{vals}) = 0.54$$

$$\text{Output}_{\text{ShearCriticalSection1}} := \frac{V_u}{\phi V_n} \left( \frac{x_{\text{critical}}}{0.5\text{ft}} + 1 \right)$$

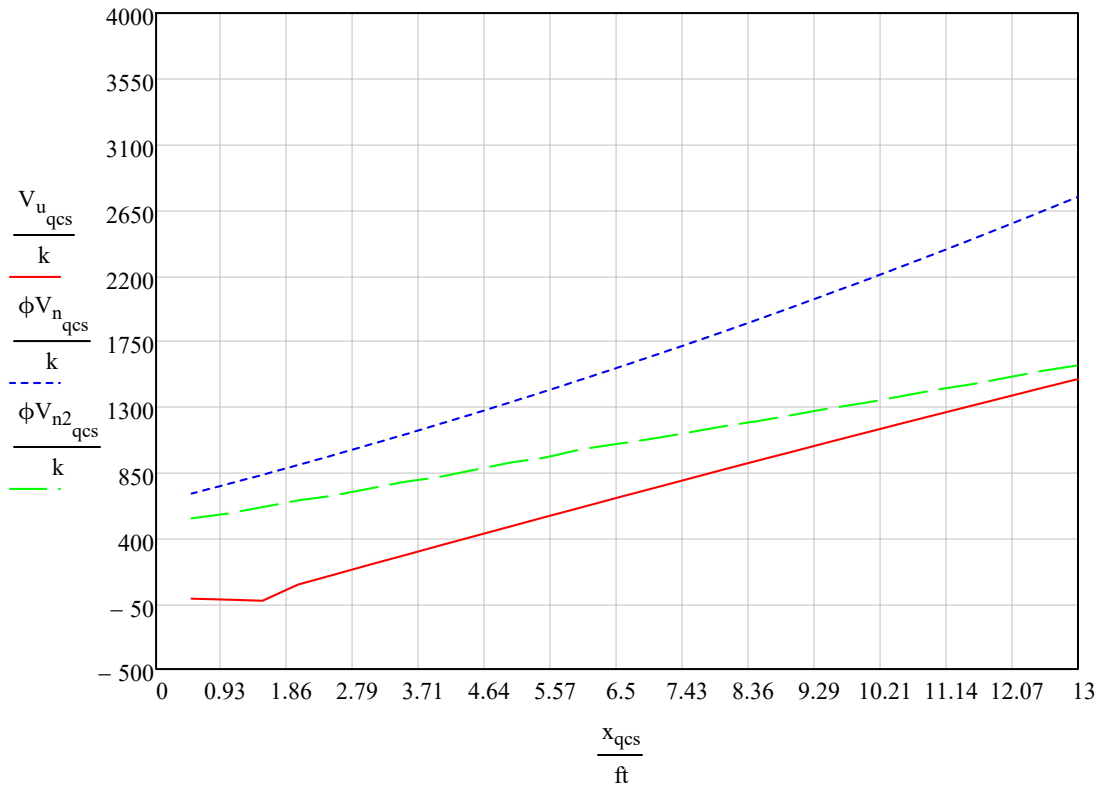
$$\text{vals}_{ii} := \frac{V_{u_{ii}}}{\phi V_{n2_{ii}}}$$

$$\text{Check}_{\text{Shear2}} := \max(\text{vals}) = 0.54$$

$$\text{Output}_{\text{ShearCriticalSection2}} := \frac{V_u}{\phi V_{n2}} \left( \frac{x_{\text{critical}}}{0.5\text{ft}} + 1 \right)$$

**Factored Shear vs.  
 Shear Capacity -**

$x_{critical} = 12.6 \text{ ft}$



### IX-e. Pedestal Two-Way Shear Capacity Check

Effective depth at face of pedestal

$$d_{\text{mid}} := h_b + h_c - cc_{\text{bot}} - di_{\text{botm}} \quad d_{\text{mid}} = 57.6 \cdot \text{in}$$

Effective depth at face of pedestal

$$d_{\text{face}} := \text{if} \left[ \frac{C}{2} + \frac{d_{\text{mid}}}{2} < \frac{B}{2}, d_{\text{mid}} \cdot h_b + \frac{\frac{D}{2} - \left( \frac{C}{2} + \frac{d_{\text{mid}}}{2} \right)}{a} \cdot h_c - cc_{\text{bot}} - di_{\text{botm}} \right]$$

$$d_{\text{face}} = 56.3 \cdot \text{in}$$

Area of critical section:

$$A_c := 2\pi \cdot d_{\text{face}} \cdot \left( \frac{C + d_{\text{face}}}{2} \right) \quad A_c = 46066 \cdot \text{in}^2$$

Polar moment of inertia  
of critical section:

$$J_c := \pi \cdot d_{\text{face}} \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^3 + \left( \frac{d_{\text{face}}^3}{3} \right) \left( \frac{C + d_{\text{face}}}{2} \right) \quad J_c = 3.98 \times 10^8 \cdot \text{in}^4$$

Perimeter of critical section:

$$b_0 := 2\pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right) \quad b_0 = 818 \cdot \text{in}$$

Half critical section width:

$$c := \frac{C + d_{\text{face}}}{2} \quad (\text{interior column}) \quad c = 130 \cdot \text{in}$$

Weight of pedestal:

$$W_p = 79 \cdot \text{kip}$$

Unfactored vertical wind  
load on critical section:

$$P_W := W_t + W_p + (h_c + h_b) \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 \right] \cdot \gamma_c \dots$$

$$+ \gamma_{\text{sdtop}} \cdot [h_s - (h_c + h_b)] \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 - \pi \cdot \left( \frac{C}{2} \right)^2 \right]$$

$$P_W = 848 \cdot \text{k}$$

Unfactored vertical seismic  
load on critical section:

$$P_{EQ} := W_{OE} + W_p + (h_c + h_b) \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 \right] \cdot \gamma_c \dots$$

$$+ \gamma_{\text{sdtop}} \cdot [h_s - (h_c + h_b)] \cdot \left[ \pi \cdot \left( \frac{C + d_{\text{face}}}{2} \right)^2 - \pi \cdot \left( \frac{C}{2} \right)^2 \right]$$

$$P_{EQ} = 851 \cdot \text{k}$$

Unbalanced wind moment on joint:

$$M_{\text{unbalancedW}} = 31968 \cdot \text{kip} \cdot \text{ft}$$

Fraction of wind moment that can  
be carried by flexure:

$$\gamma_{fW} = 0.85$$

Fraction of wind moment carried  
by shear:

$$\gamma_{vW} := \max(0.4, 1 - \gamma_{fW}) \quad \gamma_{vW} = 0.40$$

Factored shear stress due to  
wind load at critical section:

$$v_{uW} := \frac{\alpha_{d2} \cdot P_W}{A_c} + \frac{\gamma_{vW} \cdot M_{\text{unbalancedW}} \cdot c}{J_c} \quad v_{uW} = 72 \cdot \text{psi}$$

Unbalanced seismic moment on joint:  $M_{\text{unbalancedEQ}} = 15510 \cdot \text{kip} \cdot \text{ft}$

Fraction of seismic moment that can be carried by flexure:  $\gamma_{\text{fEQ}} = 1.74$

Fraction of seismic moment carried by shear:  $\gamma_{\text{vEQ}} := \max(0.4, 1 - \gamma_{\text{fEQ}})$   $\gamma_{\text{vEQ}} = 0.40$

Factored shear stress due to seismic load at critical section:  $v_{\text{uEQ}} := \frac{\alpha_{\text{d2EQ}} \cdot P_{\text{EQ}}}{A_{\text{c}}} + \frac{\gamma_{\text{vEQ}} \cdot M_{\text{unbalancedEQ}} \cdot c}{J_{\text{c}}}$   $v_{\text{uEQ}} = 47 \cdot \text{psi}$

Determine controlling load case:  $v_{\text{u}} := \max(v_{\text{uW}}, v_{\text{uEQ}})$   $v_{\text{u}} = 72 \text{ psi}$

$$\beta_{\text{c}} := 1$$

$$\alpha_{\text{s}} := 40$$

$$\phi_{\text{v}} = 0.75$$

Shear stress capacity:  $\phi v_{\text{c}} := \phi_{\text{v}} \cdot \min \left[ \left( 2 + \frac{4}{\beta_{\text{c}}} \right), \left( \frac{\alpha_{\text{s}} \cdot d_{\text{face}}}{b_0} + 2 \right), 4 \right] \cdot \sqrt{f_{\text{c}}} \cdot \text{psi}$   $\phi v_{\text{c}} = 212 \cdot \text{psi}$  (Reference 1a)

Check of factored shear stress vs. shear stress capacity:  $\frac{v_{\text{u}}}{\phi v_{\text{c}}} = 0.34$

## X. Concrete Design - Fatigue Loads

### A. Design Functions

Function describing the volume of concrete for each slice of the moment/shear calculations.

$$\text{ConcreteVolumeFat}(y) := \begin{cases} h_b \cdot (B + 2 \cdot y) + \frac{y}{a} \cdot h_c \cdot (B + y) & \text{if } y \leq a \\ h_b \cdot (D) + h_c \cdot (B + a) & \text{otherwise} \end{cases}$$

Functions describing the weight of the soil wedge pieces acting on each slice of the moment/shear calculations.

$$\text{StaticSoilWedgeWeightFat}(\gamma_{sd}, \gamma_{ss}) := \begin{cases} \gamma_{sd} \cdot \frac{B \cdot \tan(\theta_{fat})}{2} \cdot (h_s - h_b)^2 & \text{if } d_{GWTF} \geq h_s - h_b \\ \frac{B \cdot \tan(\theta_{fat})}{2} \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{otherwise} \end{cases}$$

$$\text{VariableSoilWedgeWeightFat}(y, \gamma_{sd}, \gamma_{ss}) := \begin{cases} 0 & \text{if } d_{GWTF} \geq h_s - h_b \\ \text{otherwise} \\ \begin{cases} \sqrt{2} \cdot \tan(\theta_{fat}) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{if } y \leq a \\ \tan(\theta_{fat}) \cdot \left[ \gamma_{ss} (h_s - h_b - d_{GWTF})^2 + \gamma_{sd} \left[ (h_s - h_b)^2 - (h_s - h_b - d_{GWTF})^2 \right] \right] & \text{otherwise} \end{cases} \end{cases}$$

Function describing the volume of dry soil over each slice of the moment/shear calculations.

$$\text{DrySoilVolumeFat}(h_j, y) := \begin{cases} \text{if } d_{\text{GWTF}} \geq h_s - h_b \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \frac{y^2 \cdot h_c}{a} + \sqrt{2} \cdot \tan(\theta_{\text{fat}}) \cdot (h_s - h_b)^2 \right] & \text{if } y \leq a \\ D \cdot \left[ (h_s - h_b) - h_c \right] + h_c \cdot a + \tan(\theta_{\text{fat}}) \cdot (h_s - h_b)^2 & \text{otherwise} \\ \text{if } d_{\text{GWTF}} \leq h_s - h_j \\ d_{\text{GWTF}} \cdot (B + 2 \cdot y) & \text{if } y \leq a \\ D \cdot d_{\text{GWTF}} & \text{otherwise} \\ \text{otherwise} \\ \left[ \left( (h_s - h_b) - \frac{y}{a} \cdot h_c \right) \cdot (B + 2 \cdot y) + \left[ \frac{y^2 \cdot h_c}{a} - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] & \text{if } y \leq a \\ \left[ D \cdot (h_s - h_b - h_c) + \left[ h_c \cdot a - \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 \right] \right] & \text{otherwise} \end{cases}$$

Function describing the volume of saturated soil over each slice of the moment / shear calculations.

$$\text{SaturatedSoilVolumeFat}(h_j, y) := \begin{cases} 0 & \text{if } d_{\text{GWTF}} \geq h_s - h_b \\ \text{if } d_{\text{GWTF}} \leq h_s - h_j \\ \left( (B + 2 \cdot y) \cdot \left( h_s - h_b - \frac{y}{a} \cdot h_c - d_{\text{GWTF}} \right) + \frac{y^2 \cdot h_c}{a} \right) & \text{if } y \leq a \\ (h_s - h_b - h_c - d_{\text{GWTF}}) \cdot D + h_c \cdot a & \text{otherwise} \\ \frac{a}{h_c} \cdot (h_s - h_b - d_{\text{GWTF}})^2 & \text{otherwise} \end{cases}$$

Function describing the effect of groundwater on the material weights over each slice of the moment/ shear calculations.

$$\text{BuoyancyWeightFat}(y) := \begin{cases} 0 & \text{if } d_{\text{GWTF}} \geq h_s \\ \text{if } d_{\text{GWTF}} < h_s \\ (B + 2 \cdot y) \cdot (h_s - d_{\text{GWTF}}) & \text{if } y \leq a \\ (h_s - d_{\text{GWTF}}) \cdot D & \text{otherwise} \end{cases}$$

## B. Bottom Reinforcement

(Reference 7)

Depth to reinforcement at critical section for flat portion of footing:

$$d_{\text{face2}} := h_b + h_c - cc_{\text{bot}} - 1.5d_{\text{ibotm}} - 1 \text{ in} \quad d_{\text{face2}} = 55.9 \text{ in}$$

Width of Concrete Resisting Fatigue:

$$w_f := OD + 3 \cdot (h_c + h_b)$$

$$\frac{w_f}{D} = 61\%$$

Cap width to be within "middle" strip:

$$w_f := \min(w_f, 2 \cdot W_m) \quad w_f = 30.45 \text{ ft}$$

Area of steel across critical section:

$$A_s := \frac{w_f}{s_{\text{botm}}} \cdot A_{\text{botm}} \quad A_s = 63.3 \cdot \text{in}^2$$

Footing Depth at Edge of Section:

$$d_{\text{edge}} := d_{\text{face2}} - \frac{w_f - B}{2} \cdot \frac{h_c}{a} = 44.58 \text{ in}$$

Concrete Area Resisting Shear:

$$A_{\text{face}} := B \cdot d_{\text{face2}} + 2 \cdot \frac{w_f - B}{2} \cdot \left( \frac{d_{\text{edge}} + d_{\text{face2}}}{2} \right) \quad A_{\text{face}} = 19758 \cdot \text{in}^2$$

## C. Fatigue Soil Bearing Pressure

Service load eccentricity:

$$e_{f\text{North}} := \frac{M_{\text{Unique}}{}_{\text{north}}}{W_{\text{fat}}}$$

Circular radius of octagon:

$$R := \frac{D}{2} \quad R = 25.00 \text{ ft}$$

Effective soil area in bearing:

$$A_{\text{effNorth}} := 2 \cdot \left[ \left( R^2 \right) \cdot \arccos \left( \frac{e_{f\text{North}}}{R} \right) \dots \right. \\ \left. + -e_{f\text{North}} \cdot \sqrt{R^2 - \left( e_{f\text{North}} \right)^2} \right]$$

Ellipse soil width in bearing:

$$b_{e\text{North}} := 2 \cdot \left( R - e_{f\text{North}} \right)$$

Ellipse soil length in bearing:

$$l_{e\text{North}} := 2 \cdot R \cdot \sqrt{1 - \left( 1 - \frac{b_{e\text{North}}}{2 \cdot R} \right)^2}$$

Effective soil length in bearing:

$$l_{\text{effNorth}} := \sqrt{A_{\text{effNorth}} \cdot \frac{l_{e\text{North}}}{b_{e\text{North}}}}$$



Effective soil width in bearing:

$$b_{\text{effNorth}_{q_{\text{rnorth}}}} := \frac{I_{\text{effNorth}_{q_{\text{rnorth}}}}}{I_{\text{eNorth}_{q_{\text{rnorth}}}}} \cdot b_{\text{eNorth}_{q_{\text{rnorth}}}}$$

Maximum fatigue bearing pressure:

$$f_{\text{fNorth}_{q_{\text{rnorth}}}} := \frac{W_{\text{fat}}}{A_{\text{effNorth}_{q_{\text{rnorth}}}}}$$

$$x_{\text{startNorth}_{q_{\text{rnorth}}}} := \frac{D}{2} - e_{\text{fNorth}_{q_{\text{rnorth}}}} - \frac{b_{\text{effNorth}_{q_{\text{rnorth}}}}}{2}$$

Foundation plan area:

$$A_{\text{base}} := D^2 - 2 \cdot \left( \frac{D - B}{2} \right)^2 \quad A_{\text{base}} = 2071 \text{ ft}^2$$

Section modulus of foundation for normal orientation:

$$S_{\text{normal}} := \frac{2I_{\text{fdn}}}{D} \quad S_{\text{normal}} = 13684 \cdot \text{ft}^3$$

$$W_{\text{fat}} = 1946 \cdot \text{kip}$$

$$\frac{W_{\text{fat}}}{A_{\text{base}}} = 940 \cdot \text{psf}$$

Moment at which the foundation lifts:

$$M_{\text{maxlift}} := \frac{W_{\text{fat}}}{A_{\text{base}}} \cdot S_{\text{normal}} = 12860 \cdot \text{k} \cdot \text{ft}$$

Maximum soil pressure at point when the foundation lifts:

$$\sigma_{\text{maxlift}} := \frac{W_{\text{fat}}}{A_{\text{base}}} + \frac{M_{\text{maxlift}}}{S_{\text{normal}}} = 1880 \cdot \text{psf}$$

**Min and Max soil bearing pressure for each fatigue range:**

Maximum soil pressure at point when the foundation lifts defined for each individual fatigue load:

$$\sigma_{\text{north\_max\_soiltrap}}_{qf_{\text{north}}} := \frac{W_{\text{fat}}}{A_{\text{base}}} + \frac{M_{\text{Unique}}_{\text{north}}_{qf_{\text{north}}}}{S_{\text{normal}}}$$

$$\sigma_{\text{north\_min\_soil}}_{qf_{\text{north}}} := \begin{cases} \frac{W_{\text{fat}}}{A_{\text{base}}} - \frac{M_{\text{Unique}}_{\text{north}}_{qf_{\text{north}}}}{S_{\text{normal}}} & \text{if } \frac{W_{\text{fat}}}{A_{\text{base}}} - \frac{M_{\text{Unique}}_{\text{north}}_{qf_{\text{north}}}}{S_{\text{normal}}} > 0 \\ \text{"LIFT"} & \text{otherwise} \end{cases}$$

Guess for solver of soil bearing length:  $L_b := \frac{3}{4} \cdot D$        $L_b = 37.50 \text{ ft}$

Guess for solver of max soil pressure:  $f_{\text{max}} := \sigma_{\text{maxlift}}$

The following functions solve for the soil pressure assuming the pressure distribution is triangular and lift-off has occurred on the minimum pressure side of the foundation:

$F_{\text{VALS}}_{qf_{\text{north}}} := W_{\text{fat}}$

$M_{\text{TOEVALS}}_{qf_{\text{north}}} := W_{\text{fat}} \cdot \frac{D}{2} - M_{\text{Unique}}_{\text{north}}_{qf_{\text{north}}}$

Given

$$F = \int_0^a (B + 2 \cdot y) \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy \dots$$

$$+ \int_a^{a+B} D \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy \dots$$

$$+ \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] dy$$

$$M_{\text{toe}} = \int_0^a (B + 2 \cdot y) \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy \dots$$

$$+ \int_a^{a+B} D \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy \dots$$

$$+ \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[ f_{\text{max}} - f_{\text{max}} \cdot \left( \frac{y}{L_b} \right) \right] y dy$$

$$\text{FUNCTION}(F, M_{\text{toe}}) := \text{Find} \left( \frac{L_b}{\text{ft}}, \frac{f_{\text{max}}}{\text{psf}} \right)$$

Solve the loop for the "LIFT" condition:

$$\text{Map}L_{b_{q_{r_{north}}}} := \begin{cases} \text{FUNCTION}(F_{\text{VALS}}_{q_{r_{north}}}, M_{\text{TOEVALS}}_{q_{r_{north}}}) \cdot 0 \cdot \text{ft} & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"} \\ 0 & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} \neq \text{"LIFT"} \end{cases}$$

$$\sigma_{\text{north\_max\_soiltri}}_{q_{r_{north}}} := \begin{cases} \text{FUNCTION}(F_{\text{VALS}}_{q_{r_{north}}}, M_{\text{TOEVALS}}_{q_{r_{north}}}) \cdot 1 \cdot \text{psf} & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"} \\ 0 & \text{if } \sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} \neq \text{"LIFT"} \end{cases}$$

Select the actual pressure depending on if the soil pressure at each fatigue load depending on if "lift" has occurred (triangular soil pressure distribution) or not:

$$\sigma_{\text{north\_max\_soil}}_{q_{r_{north}}} := \text{if}(\sigma_{\text{north\_min\_soil}}_{q_{r_{north}}} = \text{"LIFT"}, \sigma_{\text{north\_max\_soiltri}}_{q_{r_{north}}}, \sigma_{\text{north\_max\_soiltrap}}_{q_{r_{north}}}) = \dots$$

Soil pressure output for each fatigue load (shown in partial tabular form and graphically):

$q_{r_{north}} =$	$M_{\text{Unique}}_{\text{north}} =$	$\sigma_{\text{north\_min\_soil}} =$	$\sigma_{\text{north\_max\_soiltrap}} =$	$\sigma_{\text{north\_max\_soiltri}} =$	$\sigma_{\text{north\_max\_soil}} =$
	$\cdot \text{k} \cdot \text{ft}$	$\cdot \text{psf}$	$\cdot \text{psf}$	$\cdot \text{psf}$	$\cdot \text{psf}$
1	0	940	940	0	940
2	38	937	943	0	943
3	76	934	945	0	945
4	114	931	948	0	948
5	152	929	951	0	951
6	190	926	954	0	954
7	228	923	956	0	956
8	266	920	959	0	959
9	304	918	962	0	962
10	342	915	965	0	965
11	380	912	968	0	968
12	418	909	970	0	970
13	456	906	973	0	973
14	494	904	976	0	976
15	532	901	979	0	979
16	570	898	981	0	981
17	608	895	984	0	984
18	646	893	987	0	987
19	684	890	990	0	990
...	...	...	...	...	...

### D. Map Unique Matrix Loop Results back to full Minimum Markov or Rain Flow Matrix

$$L_{bmin_{qr}} := \sum_{q_{r,north} = 0}^{q_{t,north}} \left( \begin{array}{l} \text{Map}L_{b_{q_{r,north}}} \quad \text{if } M_{minnorth_{qr}} = M_{Unique_{north_{q_{r,north}}}} \\ 0 \quad \text{if } M_{minnorth_{qr}} \neq M_{Unique_{north_{q_{r,north}}}} \end{array} \right)$$

$$\sigma_{minnorth\_max\_soil_{qr}} := \sum_{q_{r,north} = 0}^{q_{t,north}} \left( \begin{array}{l} \sigma_{north\_max\_soil_{q_{r,north}}} \quad \text{if } M_{minnorth_{qr}} = M_{Unique_{north_{q_{r,north}}}} \\ 0 \quad \text{if } M_{minnorth_{qr}} \neq M_{Unique_{north_{q_{r,north}}}} \end{array} \right)$$

$$\text{Map}\sigma_{minnorth\_min\_soil_{qr}} := \sum_{q_{r,north} = 0}^{q_{t,north}} \left( \begin{array}{l} 0 \quad \text{if } \sigma_{north\_min\_soil_{q_{r,north}}} = \text{"LIFT"} \\ \text{otherwise} \\ \left| \begin{array}{l} \sigma_{north\_min\_soil_{q_{r,north}}} \quad \text{if } M_{minnorth_{qr}} = M_{Unique_{north_{q_{r,north}}}} \\ 0 \quad \text{if } M_{minnorth_{qr}} \neq M_{Unique_{north_{q_{r,north}}}} \end{array} \right. \end{array} \right)$$

$$\sigma_{minnorth\_min\_soil_{qr}} := \left| \begin{array}{l} \text{"LIFT"} \quad \text{if } \text{Map}\sigma_{minnorth\_min\_soil_{qr}} = 0 \\ \text{Map}\sigma_{minnorth\_min\_soil_{qr}} \quad \text{if } \text{Map}\sigma_{minnorth\_min\_soil_{qr}} \neq 0 \end{array} \right.$$

### E. Map Unique Matrix Loop Results back to full Maximum Markov or Rain Flow Matrix

$$L_{b_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} \text{Map}L_{b_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\sigma_{\text{maxnorth}_{\text{max}_{\text{soil}}}_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} \sigma_{\text{north}_{\text{max}_{\text{soil}}}_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\text{Map}\sigma_{\text{maxnorth}_{\text{min}_{\text{soil}}}_{qr}} := \sum_{q_{r_{north}} = 0}^{q_{t_{north}}} \left( \begin{array}{l} 0 \text{ if } \sigma_{\text{north}_{\text{min}_{\text{soil}}}_{q_{r_{north}}}} = \text{"LIFT"} \\ \text{otherwise} \\ \sigma_{\text{north}_{\text{min}_{\text{soil}}}_{q_{r_{north}}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{north}_{q_{r_{north}}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{north}_{q_{r_{north}}}} \end{array} \right)$$

$$\sigma_{\text{maxnorth}_{\text{min}_{\text{soil}}}_{qr}} := \begin{array}{l} \text{"LIFT"} \text{ if } \text{Map}\sigma_{\text{maxnorth}_{\text{min}_{\text{soil}}}_{qr}} = 0 \\ \text{Map}\sigma_{\text{maxnorth}_{\text{min}_{\text{soil}}}_{qr}} \text{ if } \text{Map}\sigma_{\text{maxnorth}_{\text{min}_{\text{soil}}}_{qr}} \neq 0 \end{array}$$

### F. Fatigue Load Bottom Moments and Top Moments at Critical Section

$$\begin{aligned}
 M_{fminbotNorth_{qr}} := & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face} \\
 & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{face} - y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot D \cdot (x_{face} - y) \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{face} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot x_{face}
 \end{aligned}$$

$$\begin{aligned}
 M_{f_{\max} \text{botNorth}_{qr}} := & \int_0^a \left[ \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \frac{y}{L_b}_{qr} \cdot \left( \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{\text{face}} - y) \, dy \dots && \text{if } \sigma_{\max \text{north}_{\min} \text{soil}_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{\text{face}}} \left[ \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \frac{y}{L_b}_{qr} \cdot \left( \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} \right) \right] \cdot D \cdot (x_{\text{face}} - y) \, dy \dots \\
 & + - \int_0^{x_{\text{face}}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{sdbot}} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{\text{face}} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot x_{\text{face}} \\
 & \int_0^a \left[ \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \frac{y}{D} \cdot \left( \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \sigma_{\max \text{north}_{\min} \text{soil}_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (x_{\text{face}} - y) \, dy \dots && \text{otherwise} \\
 & + \int_a^{x_{\text{face}}} \left[ \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \frac{y}{D} \cdot \left( \sigma_{\max \text{north}_{\max} \text{soil}_{qr}} - \sigma_{\max \text{north}_{\min} \text{soil}_{qr}} \right) \right] \cdot D \cdot (x_{\text{face}} - y) \, dy \dots \\
 & + - \int_0^{x_{\text{face}}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{sdbot}} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \\ & + -\text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (x_{\text{face}} - y) \, dy \dots \\
 & + -\text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot x_{\text{face}}
 \end{aligned}$$

$$\begin{aligned}
 M_{f_{\text{mintopNorth}}_{\text{qr}}} := & \max \left[ 0 \text{ k}\cdot\text{ft}, \int_a^{D-x_{\text{face\_alt}}} \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{L_{\text{bmin}}_{\text{qr}}} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} \right) \right] \cdot D \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \right. \\
 & + \int_0^a \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{L_{\text{bmin}}_{\text{qr}}} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \int_0^{D-x_{\text{face\_alt}}} \left[ \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \right. \\
 & \quad \left. + \text{DrySoilVolumeFat}(h_b + h_c, y) \gamma_{\text{sdbot}} \dots \right. \\
 & \quad \left. + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \right. \\
 & \quad \left. + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \right. \\
 & \quad \left. + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \right] \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot (D - x_{\text{face\_alt}}) \dots \\
 & + -W_{\text{mean}} \cdot \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c)(h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c - \gamma_{\text{sdbot}})(h_p - h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -M_{\text{minnorth}}_{\text{qr}} \left. \right] \text{ if } \sigma_{\text{min}} \\
 \\
 & \max \left[ 0 \text{ k}\cdot\text{ft}, \int_0^a \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{D} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \sigma_{\text{minnorth\_min\_soil}}_{\text{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \right. \\
 & + \int_a^{D-x_{\text{face\_alt}}} \left[ \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \frac{y}{D} \cdot \left( \sigma_{\text{minnorth\_max\_soil}}_{\text{qr}} - \sigma_{\text{minnorth\_min\_soil}}_{\text{qr}} \right) \right] \cdot D \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \int_0^{D-x_{\text{face\_alt}}} \left[ \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \right. \\
 & \quad \left. + \text{DrySoilVolumeFat}(h_b + h_c, y) \gamma_{\text{sdbot}} \dots \right. \\
 & \quad \left. + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{\text{ssbot}} \dots \right. \\
 & \quad \left. + \text{VariableSoilWedgeWeightFat}[y, \gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \dots \right. \\
 & \quad \left. + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \right] \cdot (D - x_{\text{face\_alt}} - y) \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{\text{sdbot}}, (\gamma_{\text{ssbot}} - \gamma_w)] \cdot (D - x_{\text{face\_alt}}) \dots \\
 & + -W_{\text{mean}} \cdot \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c)(h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots \\
 & + -(\gamma_c - \gamma_{\text{sdbot}})(h_p - h_{\text{pe}}) \cdot \left( \frac{\pi \cdot C^2}{4} \right) \frac{S_{\text{embedding}}}{2} \dots
 \end{aligned}$$



$$M_{fmaxtopNorth_{qr}} := \left[ \begin{aligned} & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \int_a^{D-x_{face\_alt}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} \right) \right] \cdot D \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \int_0^{D-x_{face\_alt}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot (D - x_{face\_alt}) \dots \\ & + -W_{mean} \cdot \frac{S_{embedding}}{2} \dots \\ & + -(\gamma_c)(h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ & + -(\gamma_c - \gamma_{sdbot})(h_p - h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ & + -M_{maxnorth_{qr}} \end{aligned} \right] \quad \text{if } \sigma_{maxnorth\_min\_soil}$$

$$\left[ \begin{aligned} & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \int_a^{D-x_{face\_alt}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}} \right) \right] \cdot D \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \int_0^{D-x_{face\_alt}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \cdot (D - x_{face\_alt} - y) \, dy \dots \\ & + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \cdot (D - x_{face\_alt}) \dots \\ & + -W_{mean} \cdot \frac{S_{embedding}}{2} \dots \\ & + -(\gamma_c)(h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \\ & + -(\gamma_c - \gamma_{sdbot})(h_p - h_{pe}) \cdot \left( \frac{\pi}{4} \cdot C^2 \right) \frac{S_{embedding}}{2} \dots \end{aligned} \right] \quad \text{otherwise}$$

**G. Fatigue Load Shear at Critical Section**

$$\begin{aligned}
 V_{EdminNorth_{qr}} := & \max \left( 0 \text{ kip}, \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \right. \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} \right) \right] \cdot D \, dy \dots \\
 & + \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + \text{StaticSoilWedgeWeightFat}[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \\
 & \int_0^a \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left( \sigma_{minnorth\_max\_soil_{qr}} - \sigma_{minnorth\_min\_soil_{qr}} \right) \right] \cdot D \, dy \dots \\
 & + \int_0^{x_{face}} \left[ \begin{aligned} & \text{ConcreteVolumeFat}(y) \cdot \gamma_c \dots \\ & + \text{DrySoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + \text{SaturatedSoilVolumeFat}(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + \text{VariableSoilWedgeWeightFat}[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + \text{BuoyancyWeightFat}(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots
 \end{aligned}$$

$$\begin{aligned}
 V_{EdmaxNorth_{qr}} := & \left[ + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \right] \\
 & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{if } \sigma_{maxnorth\_min\_soil_{qr}} = \text{"LIFT"} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot (\sigma_{maxnorth\_max\_soil_{qr}}) \right] \cdot D \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & ConcreteVolumeFat(y) \cdot \gamma_c \dots \\ & + DrySoilVolumeFat(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + SaturatedSoilVolumeFat(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + VariableSoilWedgeWeightFat[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -BuoyancyWeightFat(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \\
 & \int_0^a \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot (B + 2 \cdot y) \, dy \dots \quad \text{otherwise} \\
 & + \int_a^{x_{face}} \left[ \sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot (\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}) \right] \cdot D \, dy \dots \\
 & + - \int_0^{x_{face}} \left[ \begin{aligned} & ConcreteVolumeFat(y) \cdot \gamma_c \dots \\ & + DrySoilVolumeFat(h_b + h_c, y) \cdot \gamma_{sdbot} \dots \\ & + SaturatedSoilVolumeFat(h_b + h_c, y) \cdot \gamma_{ssbot} \dots \\ & + VariableSoilWedgeWeightFat[y, \gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)] \dots \\ & + -BuoyancyWeightFat(y) \cdot \gamma_w \end{aligned} \right] \, dy \dots \\
 & + -StaticSoilWedgeWeightFat[\gamma_{sdbot}, (\gamma_{ssbot} - \gamma_w)]
 \end{aligned}$$

### H. Shear and Moment Summary

Results in partial tabular form:

qr =	$V_{EdminNorth} =$	$V_{EdmaxNorth} =$	$M_{fminbotNorth} =$	$M_{fmaxbotNorth} =$	$M_{fmintopNorth} =$	$M_{fmaxtopNorth} =$
	·kip	·kip	·k·ft	·k·ft	·k·ft	·k·ft
0	159	159	1240	1240	0	-1405
1	159	159	1240	1240	0	-1405
2	159	159	1240	1240	0	-1405
3	159	159	1240	1240	0	-1405
4	159	159	1240	1240	0	-1405
5	159	159	1240	1240	0	-1405
6	159	159	1240	1240	0	-1405
7	159	159	1240	1240	0	-1405
8	159	159	1240	1240	0	-1405
9	159	159	1240	1240	0	-1405
10	159	159	1240	1240	0	-1405
11	159	159	1240	1240	0	-1405
12	159	159	1240	1240	0	-1405
13	159	159	1240	1240	0	-1405
14	159	159	1240	1240	0	-1405
15	159	159	1240	1240	0	-1405
16	159	159	1240	1240	0	-1405
17	159	159	1240	1240	0	-1405
18	159	159	1240	1240	0	-1405
19	159	159	1240	1240	0	-1405
20	159	159	1240	1240	0	-1405
21	159	159	1240	1240	0	-1405
22	159	159	1240	1240	0	-1405
23	159	159	1240	1240	0	-1405
24	159	159	1240	1240	0	-1405
25	159	159	1240	1240	0	-1405
26	159	159	1240	1240	0	-1405
27	159	159	1240	1240	0	-1405
...	...	...	...	...	...	...

## I. Transformed Section Analysis at Critical Section

The neutral axis depth in the cracked section is governed by the following cubic equation:

$$\text{CUBIC} := 0$$

Given  $x_{cr} := 11.11 \cdot \text{in}$

$$\text{CUBIC} = \frac{a \cdot x_{cr}^3}{3 \cdot h_c} + \frac{B \cdot x_{cr}^2}{2} - n_{mod} \cdot A_s \cdot (d_{face2} - x_{cr})$$

$$x_{cr} := \text{Find}(x_{cr}) \quad x_{cr} = 12.92 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{CR} := \frac{B \cdot x_{cr}^3}{3} + \frac{2 \cdot \left(\frac{x_{cr} \cdot a}{h_c}\right) \cdot x_{cr}^3}{12} + n_{mod} \cdot A_s \cdot (d_{face2} - x_{cr})^2$$

$$I_{CR} = 60.5 \cdot \text{ft}^4$$

## J. Compute Concrete and Steel Stresses

Elastic beam theory prediction of minimum compressive stress in concrete:

$$\sigma_{cminNorth_{qr}} := \max\left(0 \text{ psi}, \frac{M_{fminbotNorth_{qr}} \cdot x_{cr}}{I_{CR}}\right)$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stminNorth_{qr}} := \max\left[0 \text{ psi}, \frac{n_{mod} \cdot M_{fminbotNorth_{qr}} \cdot (d_{face2} - x_{cr})}{I_{CR}}\right]$$

Elastic beam theory prediction of compressive stress in concrete:

$$\sigma_{cmaxNorth_{qr}} := \frac{M_{fmaxbotNorth_{qr}} \cdot x_{cr}}{I_{CR}}$$

Elastic beam theory prediction of tensile stress in reinforcement:

$$\sigma_{stmaxNorth_{qr}} := \frac{n_{mod} \cdot M_{fmaxbotNorth_{qr}} \cdot (d_{face2} - x_{cr})}{I_{CR}}$$

### K. Compute Twisting Moments and Steel Stresses

Map results from unique matrices back to full matrices:

$$b_{\text{effminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} b_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$x_{\text{start\_minNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} x_{\text{startNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$f_{\text{fminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} f_{\text{fNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

$$l_{\text{effminNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} l_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{minnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \\ 0 \text{ if } M_{\text{minnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}}} \end{array} \right)$$

Minimum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{\text{twist\_min}_{qr}} := \min(b_{\text{effminNorth}_{qr}}, a - x_{\text{start\_minNorth}_{qr}}) \cdot f_{\text{fminNorth}_{qr}} \cdot \frac{\left( \frac{l_{\text{effminNorth}_{qr}} - C}{2} \right)^2}{2}$$

Map results from unique matrices back to full matrices:

$$b_{\text{effmaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} b_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}} \end{array} \right)$$

$$x_{\text{start\_maxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} x_{\text{startNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}} \end{array} \right)$$

$$f_{\text{fimaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} f_{\text{fNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}} \end{array} \right)$$

$$l_{\text{effimaxNorth}_{qr}} := \sum_{q_{r\text{north}} = 0}^{q_{t\text{north}}} \left( \begin{array}{l} l_{\text{effNorth}_{qr\text{north}}} \text{ if } M_{\text{maxnorth}_{qr}} = M_{\text{Unique}_{\text{north}_{qr\text{north}}} \\ 0 \text{ if } M_{\text{maxnorth}_{qr}} \neq M_{\text{Unique}_{\text{north}_{qr\text{north}}} \end{array} \right)$$

Maximum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{\text{twist\_max}_{qr}} := \min(b_{\text{effmaxNorth}_{qr}}, a - x_{\text{start\_maxNorth}_{qr}}) \cdot f_{\text{fimaxNorth}_{qr}} \cdot \frac{\left( \frac{l_{\text{effimaxNorth}_{qr}} - C}{2} \right)^2}{2}$$

Area of steel available in sloped foundation section to transfer twisting moments:

$$A_{s\_twist} := \left( \frac{W_m - \frac{B}{2}}{s_{botm}} \right) \cdot (A_{botm}) + \left( \frac{\frac{D}{2} - W_m}{s_{boto}} \right) \cdot (A_{boto}) = 19.3 \cdot \text{in}^2$$

Depth to reinforcement at critical section for flat portion of footing:

$$d_{face\_twist} := h_b + h_c - cc_{bot} - di_{botm} \quad d_{face\_twist} = 57.6 \cdot \text{in}$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

$$\text{QUBIC} := 0$$

Given  $x_{cr2} := 19.54 \cdot \text{in}$

$$\text{QUBIC} = \frac{2x_{cr2}}{3} \cdot \frac{x_{cr2}}{2} \cdot \frac{x_{cr2} \cdot a}{h_c} - n_{mod} \cdot A_{s\_twist} \cdot d_{face\_twist}$$

$$x_{cr2} := \text{Find}(x_{cr2}) \quad x_{cr2} = 17.96 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{CR\_twist} := \frac{\left( x_{cr2} \cdot \frac{a}{h_c} \right) \cdot x_{cr2}^3}{12} + n_{mod} \cdot A_{s\_twist} \cdot (d_{face\_twist} - x_{cr2})^2$$

$$I_{CR\_twist} = 15.3 \cdot \text{ft}^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stmin\_twist\_qr} := \max \left[ 0 \text{psi}, \frac{n_{mod} \cdot M_{twist\_min\_qr} \cdot (d_{face\_twist} - x_{cr2})}{I_{CR\_twist}} \right]$$

Elastic beam theory prediction of tensile stress in reinforcement:

$$\sigma_{stmax\_twist\_qr} := \frac{n_{mod} \cdot M_{twist\_max\_qr} \cdot (d_{face\_twist} - x_{cr2})}{I_{CR\_twist}}$$



### L. Concrete and Flexural Steel Stress Summary

Results in partial tabular form:

qr =	$\sigma_{cminNorth} =$	$\sigma_{cmaxNorth} =$	$\sigma_{stminNorth} =$	$\sigma_{stmaxNorth} =$	$\sigma_{stmin\_twist} =$	$\sigma_{stmax\_twist} =$
0	153 psi	153 psi	4585 psi	4585 psi	14722 psi	14722 psi
1	153	153	4585	4585	14722	14722
2	153	153	4585	4585	14722	14722
3	153	153	4585	4585	14722	14722
4	153	153	4585	4585	14722	14722
5	153	153	4585	4585	14722	14722
6	153	153	4585	4585	14722	14722
7	153	153	4585	4585	14722	14722
8	153	153	4585	4585	14722	14722
9	153	153	4585	4585	14722	14722
10	153	153	4585	4585	14722	14722
11	153	153	4585	4585	14722	14722
12	153	153	4585	4585	14722	14722
13	153	153	4585	4585	14722	14722
14	153	153	4585	4585	14722	14722
15	153	153	4585	4585	14722	14722
16	153	153	4585	4585	14722	14722
17	153	153	4585	4585	14722	14722
18	153	153	4585	4585	14722	14722
19	153	153	4585	4585	14722	14722
20	153	153	4585	4585	14722	14722
21	153	153	4585	4585	14722	14722
22	153	153	4585	4585	14722	14722
23	153	153	4585	4585	14722	14722
24	153	153	4585	4585	14722	14722
25	153	153	4585	4585	14722	14722
26	153	153	4585	4585	14722	14722
27	153	153	4585	4585	14722	14722
28	153	153	4585	4585	14722	14722
29	153	153	4585	4585	14722	14722
...	...	...	...	...	...	...

### M. Check of Fatigue in Concrete due to Compressive Stress

Material Coefficient for Reinforced Concrete:

$$\gamma_{cdnv} := 1.35$$

Characteristic Compressive Cylinder Strength:

$$f_{ck} := f_{ck\_conversion} \cdot f_c \quad f_{ck} = 32.8 \cdot \text{MPa}$$

Normalized Structural Compressive Strength:

$$f_{cn} := f_{ck} \cdot \left( 1 - \frac{f_{ck}}{600 \cdot \text{MPa}} \right) \quad f_{cn} = 31.0 \cdot \text{MPa}$$

Design Concrete Compressive Strength:

$$f_{cd} := \frac{f_{cn}}{\gamma_{cdnv}} \quad f_{cd} = 22.9 \cdot \text{MPa}$$

Ratio between smallest and largest stresses in compression zone:

$$\beta := \text{if}(x_{cr} > 300 \text{mm}, 1, 0) \quad \beta = 1.00$$

Amplification factor to for linear stress distribution in compression zone:

$$\alpha := \max(1.0, 1.3 - 0.3 \cdot \beta) \quad \alpha = 1.00$$

Compressive Strength for Fatigue Check:

$$f_{rd} := \alpha \cdot f_{cd} \quad f_{rd} = 22.9 \cdot \text{MPa}$$

Exposure Factor:

$$C_{1dnv} := 12$$

Fatigue Strength Parameter:

$$C_{5c} := 1.0$$

Cycles Allowed:

$$n_{callowNorth_{qr}} := 10 \cdot C_{1dnv} \cdot \left( \frac{1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}} \right)$$

$$X_{dnvNorth_{qr}} := \frac{C_{1dnv}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2dnvNorth_{nr}} := \max \left[ 1 + 0.2 \cdot \left( \log(n_{callowNorth_{nr}}) - X_{dnvNorth_{nr}} \right), 1.0 \right]$$

$$n_{callowNorth_{qr}} := \text{if} \left[ \log(n_{callowNorth_{qr}}) > X_{dnvNorth_{qr}}, 10 \cdot C_{1dnv} \cdot C_{2dnvNorth_{qr}} \cdot \left( \frac{1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}} \right), n_{callowNorth_{qr}} \right]$$

Damage:

$$\text{Damage}_{North_{qr}} := \frac{N_{fat_{qr}}}{n_{callowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108):

$$\text{Damage}_{totalNorth} := \sum \text{Damage}_{North}$$

$$\text{Damage}_{totalNorth} = 0.00$$

$$\text{if}(\text{Damage}_{totalNorth} \leq 0.33, \text{"OK"}, \text{"NG"}) = \text{"OK"}$$

## N. Check of Fatigue in Concrete due to Shear

Concrete Area Resisting Shear:  $A_{\text{face}} = 137 \text{ ft}^2$

Characteristic Tensile Strength:  $f_{\text{tk}} := 0.48 \cdot (f_{\text{cck}} \cdot \text{MPa})^{0.5}$   $f_{\text{tk}} = 2.75 \cdot \text{MPa}$

Normalized Tensile Strength:  $f_{\text{tn}} := f_{\text{tk}} \cdot \left[ 1 - \left( \frac{f_{\text{tk}}}{25 \cdot \text{MPa}} \right)^{0.6} \right]$   $f_{\text{tn}} = 2.02 \cdot \text{MPa}$

Design Tensile Strength:  $f_{\text{td}} := \frac{f_{\text{tn}}}{\gamma_{\text{cdnv}}}$   $f_{\text{td}} = 1.49 \cdot \text{MPa}$

Design Constants:  $k_A := 100 \cdot \text{MPa}$

$d_1 := 1000 \cdot \text{mm}$

Anchored Reinforcement on Tensile Side:  $A_s = 63 \cdot \text{in}^2$

Design Factor:  $k_v := \min \left( \max \left( 1.5 - \frac{d_{\text{face}2}}{d_1}, 1.0 \right), 1.4 \right)$   $k_v = 1.00$

Design Shear Strength:  $V_{\text{cd}} := \min \left[ 0.3 \cdot \left( f_{\text{td}} + \frac{k_A \cdot A_s}{\gamma_{\text{cdnv}} \cdot A_{\text{face}}} \right) \cdot A_{\text{face}} \cdot k_v, 0.6 \cdot f_{\text{td}} \cdot A_{\text{face}} \cdot k_v \right]$

$V_{\text{cd}} = 1488 \cdot \text{k}$

Design Shear Strength Stated in Terms of Stress:  $v_{\text{cd}} := \min \left[ 0.3 \cdot \left( f_{\text{td}} + \frac{k_A \cdot A_s}{\gamma_{\text{cdnv}} \cdot A_{\text{face}}} \right) \cdot k_v, 0.6 \cdot f_{\text{td}} \cdot k_v \right]$   $v_{\text{cd}} = 75 \text{ psi}$

Cycles Allowed:  $n_{\text{vallowNorth}_{\text{qr}}} := 10$

$$C_{1\text{dnv}} \cdot \frac{\left( 1 - \frac{V_{\text{EdmaxNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} \right)}{\left( 1 - \frac{V_{\text{EdminNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} \right)}$$

$$X_{\text{dnvNorth}_{\text{qr}}} := \frac{C_{1\text{dnv}}}{1 - \frac{V_{\text{EdminNorth}_{\text{qr}}}}{C_{5c} \cdot V_{\text{cd}}} + 0.1 \cdot C_{1\text{dnv}}}$$

$$C_{2\text{dnvNorth}_{\text{qr}}} := \max \left[ 1 + 0.2 \cdot \left( \log \left( n_{\text{vallowNorth}_{\text{qr}}} \right) - X_{\text{dnvNorth}_{\text{qr}}} \right), 1.0 \right]$$

$$n_{vallowNorth_{qr}} := \text{if} \left[ \log \left( n_{vallowNorth_{qr}} \right) > X_{dvnNorth_{qr}}, 10, n_{vallowNorth_{qr}} \right]$$

$$C_{1dvn} \cdot C_{2dvnNorth_{qr}} \cdot \frac{\left( \frac{V_{EdmaxNorth_{qr}}}{1 - \frac{V_{EdmaxNorth_{qr}}}{C_{5c} \cdot V_{cd}}} \right)}{\left( \frac{V_{EdminNorth_{qr}}}{1 - \frac{V_{EdminNorth_{qr}}}{C_{5c} \cdot V_{cd}}} \right)}$$

Damage:

$$Damage_{vNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{vallowNorth_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$Damage_{vtotalNorth} := \sum Damage_{vNorth}$$

$$Damage_{vtotalNorth} = 0.00$$

$$\text{if} (Damage_{vtotalNorth} \leq 0.33, "OK", "NG") = "OK"$$

## O. Check of Fatigue in Grout Bearing Stress

Material Coefficient for Plain Grout:  $\gamma_{gdnv} := 1.35$

Exposure Factor:  $C_{1gdnv} := 12$

Fatigue Strength Parameter:  $C_{5g} := 0.8$

Characteristic Compressive Grout Strength:  $f_{grtk} := f_{cck\_conversion} \cdot f_{c28}$   $f_{grtk} = 52.4 \cdot \text{MPa}$

Normalized Structural Compressive Grout Strength:  $f_{gn} := f_{grtk} \cdot \left(1 - \frac{f_{grtk}}{600 \cdot \text{MPa}}\right)$   $f_{gn} = 47.8 \cdot \text{MPa}$

Design Concrete Compressive Grout Strength:  $f_{gd} := \frac{f_{gn}}{\gamma_{gdnv}}$   $f_{gd} = 35.4 \cdot \text{MPa}$

Anchor Bolt Pretension Load:  $T_{pre} = 72 \cdot \text{kip}$

Flange Outside Diameter:  $OD = 4556 \cdot \text{mm}$

Flange Inside Diameter:  $ID = 4000 \cdot \text{mm}$

Number of Bolts:  $N = 140$

PVC outside diameter:  $d_{SDR} = 1.660 \cdot \text{in}$

Bearing area at top of grout:  $A_{grt} := \frac{\pi}{4} \cdot (OD^2 - ID^2) - N \cdot \frac{\pi \cdot d_{SDR}^2}{4}$   $A_{grt} = 5488 \cdot \text{in}^2$

Section modulus at top of grout:  $S_{grt} := \frac{\pi}{32 \cdot OD} (OD^4 - ID^4) + \left[ \frac{2}{OD} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \right]$   $S_{grt} = 217939 \cdot \text{in}^3$

Minimum grout fatigue stress due to wind:  $b_{fat\_grt\_minnorth\_qr} := \frac{(M_{fatminnorth\_qr})}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$

Maximum grout fatigue stress due to wind:  $b_{fat\_grt\_maxnorth\_qr} := \frac{(M_{fatmaxnorth\_qr})}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$

Cycles Allowed:

$$n_{brg\_grt\_allowNorth_{qr}} := 10 \cdot C_{1gdnv} \cdot \frac{\left( \frac{b_{fat\_grt\_maxnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}{\left( \frac{b_{fat\_grt\_minnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}$$

$$X_{brg\_grt\_dnvNorth_{qr}} := \frac{C_{1gdnv}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}} + 0.1 \cdot C_{1gdnv}}$$

$$C_{2brg\_grt\_dnvNorth_{qr}} := \max \left[ 1 + 0.2 \cdot \left( \log \left( n_{brg\_grt\_allowNorth_{qr}} \right) - X_{brg\_grt\_dnvNorth_{qr}} \right), 1.0 \right]$$

$$n_{brg\_grt\_allowNorth_{qr}} := \text{if} \left[ \log \left( n_{brg\_grt\_allowNorth_{qr}} \right) > X_{brg\_grt\_dnvNorth_{qr}}, 10 \cdot C_{1gdnv} \cdot C_{2brg\_grt\_dnvNorth_{qr}} \cdot \frac{\left( \frac{b_{fat\_grt\_maxnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}{\left( \frac{b_{fat\_grt\_minnorth_{qr}}}{1 - \frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g} \cdot f_{gd}}} \right)}, n_{brg\_grt\_allowNorth_{qr}} \right]$$

Damage:

$$\text{Damage}_{brg\_grt\_North_{qr}} := \frac{N_{fat_{qr}}}{n_{brg\_grt\_allowNorth_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$\text{Damage}_{brg\_grt\_totalNorth} := \sum \text{Damage}_{brg\_grt\_North}$$

$$\text{Damage}_{brg\_grt\_totalNorth} = 0.00$$

$$\text{if} \left( \text{Damage}_{brg\_grt\_totalNorth} \leq 1.0, \text{"OK"}, \text{"NG"} \right) = \text{"OK"}$$

## P. Check of Fatigue in Pedestal Bearing Stress

Characteristic Compressive  
Cylinder Strength:

$$f_{cckp} := f_{cck\_conversion} \cdot f_{cp} \quad f_{cckp} = 32.8 \cdot \text{MPa}$$

Normalized Structural  
Compressive Strength:

$$f_{cnp} := f_{cckp} \cdot \left( 1 - \frac{f_{cckp}}{600 \cdot \text{MPa}} \right) \quad f_{cnp} = 31.0 \cdot \text{MPa}$$

Design Concrete  
Compressive Strength:

$$f_{cdp} := \frac{f_{cnp}}{\gamma_{cdnv}} \quad f_{cdp} = 22.9 \cdot \text{MPa}$$

Grout thickness:

$$t_g = 0.00 \cdot \text{in}$$

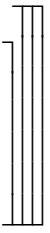
Bearing area at bottom of grout:

$$A_1 := \frac{\pi}{4} \cdot \left[ (OD + t_g)^2 - (ID - t_g)^2 \right] - N \cdot \frac{\pi \cdot d_{SDR}^2}{4} \quad A_1 = 5488 \cdot \text{in}^2$$

Section modulus at bottom of grout:

$$S_1 := \frac{\pi}{32 \cdot (OD + t_g)} \left[ (OD + t_g)^4 - (ID - t_g)^4 \right] \dots$$

$$+ \left[ \frac{2}{(OD + t_g)} \cdot \left[ N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \sum_{\lambda=1}^{\frac{N}{4}} \left[ \frac{D_i}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \dots \right. \right. \right]$$

$$\left. \left. + \left[ \frac{D_o}{2} \cdot \cos \left[ \frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1) \right] \right]^2 \right] \right] \quad S_1 = 217939 \cdot \text{in}^3$$


Minimum fatigue stress due to wind:

$$b_{fatminnorth_{qr}} := \frac{(M_{fatminnorth_{qr}})}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$$

Maximum fatigue stress due to wind:

$$b_{fatmaxnorth_{qr}} := \frac{(M_{fatmaxnorth_{qr}})}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$$

Bearing strip (radial) at bottom of grout:

$$A_{strip\_1} := \frac{(OD + t_g) - (ID - t_g)}{2} \quad A_{strip\_1} = 10.9 \cdot \text{in}$$

Angle of bearing within concrete:

$$\alpha_{DNV} := 0 \text{ deg}$$

$$\alpha_{DNV} = 0.0 \text{ deg}$$

Critical bearing angle within concrete that defines the pedestal bottom edge:

$$\alpha_{critical} := \min \left[ \text{atan} \left[ \frac{C - (OD + t_g)}{2} \right], \text{atan} \left( \frac{1}{2} \right) \right]$$

$$\alpha_{critical} = 23.7 \text{ deg}$$

Bearing strip (radial) within concrete at base of pedestal:

$$A_{strip\_2} := \min(A_{strip\_1} + 2 \cdot h_p \cdot \tan(\min(\alpha_{DNV}, \alpha_{critical})), 4 \cdot A_{strip\_1}, A_{strip\_1} + h_p)$$

$$A_{strip\_2} = 10.9 \text{ in}$$

Design Bearing Capacity Stated in Terms of Stress:

$$F_{cd} := f_{cdp} \cdot \min \left[ \left( \frac{A_{strip\_2}}{A_{strip\_1}} \right)^{\frac{1}{3}}, 1.3 \right]$$

$$F_{cd} = 3326 \text{ psi}$$

Cycles Allowed:

$$C_{1dnv} := \frac{\left( 1 - \frac{b_{fatmaxnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}{\left( 1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}$$

$$n_{brgallowNorth\_qr} := 10$$

$$X_{brgdnvNorth\_qr} := \frac{C_{1dnv}}{1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2brgdnvNorth\_qr} := \max \left[ 1 + 0.2 \cdot (\log(n_{brgallowNorth\_qr}) - X_{brgdnvNorth\_qr}), 1.0 \right]$$

$$n_{brgallowNorth\_qr} := \text{if} \left[ \log(n_{brgallowNorth\_qr}) > X_{brgdnvNorth\_qr}, 10, C_{1dnv} \cdot C_{2brgdnvNorth\_qr} \cdot \frac{\left( 1 - \frac{b_{fatmaxnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}{\left( 1 - \frac{b_{fatminnorth\_qr}}{C_{5c} \cdot F_{cd}} \right)}, n_{brgallowNorth\_qr} \right]$$

Damage:

$$\text{Damage}_{brgNorth\_qr} := \frac{N_{fat\_qr}}{n_{brgallowNorth\_qr}}$$

Accumulated Damage (Section 6, M108):

$$\text{Damage}_{brgtotalNorth} := \sum \text{Damage}_{brgNorth}$$

$$\text{Damage}_{brgtotalNorth} = 0.41$$

$$\text{if}(\text{Damage}_{brgtotalNorth} \leq 0.33, \text{"OK"}, \text{"NG"}) = \text{"NG"}$$



## R. Check of Fatigue in Bottom Steel due to Tensile Stress - Primary Direction

Design factors:  $C_3 = 19.60$

$$C_4 = 6.00$$

Tensile stress range in steel:  $\Delta\sigma_{stNorth_{qr}} := \max\left[1 \cdot \text{psi}, \left(\sigma_{stmaxNorth_{qr}} - \sigma_{stminNorth_{qr}}\right)\right]$

Characteristic strength of reinforcement:  $f_{sk} := f_{yB}$

Material coefficient for reinforcement:  $\gamma_s = 1.00$

Check maximum tensile stress in steel:  $\text{CheckStress}_{Northmax2} := \text{if}\left(\max(\sigma_{stmaxNorth}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"} , \text{"Okay"}\right) = \text{"Okay"}$

Cycles Allowed:  $n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{stNorth_{qr}}}{\text{MPa}}\right)\right)}$

$$n_{sallowNorth_{qr}} := \text{if}\left(n_{sallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:  $\text{Damage}_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$

Accumulated Damage (Section 6, M108):  $\text{Damage}_{stotalNorth} := \sum \text{Damage}_{sNorth}$

$$\text{Damage}_{stotalNorth} = 0.00$$

$$\text{if}\left(\text{Damage}_{stotalNorth} \leq 0.33, \text{"OK"} , \text{"NG"}\right) = \text{"OK"}$$

### S. Check of Fatigue in Bottom Steel due to Tensile Stress - Normal Direction

Design factors:  $C_3 = 19.60$

$C_4 = 6.00$

Tensile stress range in steel:  $\Delta\sigma_{stNorth\_normal\_qr} := \max\left[1 \cdot \text{psi}, \left(\sigma_{stmax\_twist\_qr} - \sigma_{stmin\_twist\_qr}\right)\right]$

Characteristic strength of reinforcement:  $f_{sk} := f_{yB}$

Material coefficient for reinforcement:  $\gamma_s = 1.00$

Check maximum tensile stress in steel:  $\text{CheckStress}_{Northmax3} := \text{if}\left(\max(\sigma_{stmax\_twist}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"} , \text{"Okay"}\right) = \text{"Okay"}$

Cycles Allowed:  $n_{sallowNorth\_qr} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{stNorth\_normal\_qr}}{\text{MPa}}\right)\right)}$

$n_{sallowNorth\_qr} := \text{if}\left(n_{sallowNorth\_qr} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth\_qr}\right)$

Damage:  $\text{Damage}_{sNorth\_qr} := \frac{N_{fat\_qr}}{n_{sallowNorth\_qr}}$

Accumulated Damage (Section 6, M108):  $\text{Damage}_{stotalNorth\_normal} := \sum \text{Damage}_{sNorth}$

$\text{Damage}_{stotalNorth\_normal} = 0.00$

$\text{if}\left(\text{Damage}_{stotalNorth\_normal} \leq 0.33, \text{"OK"} , \text{"NG"}\right) = \text{"OK"}$

### T. Check of Fatigue in Top Steel due to Tensile Stress

Depth to reinforcement at critical section for flat portion of footing:

$$d_{\text{face2}} := h_b + h_c - cc_{\text{top}} - 1.5d_{\text{topm}} - 1 \text{ in} \quad d_{\text{face2}} = 57.9 \cdot \text{in}$$

Depth to reinforcement at critical section for sloped portion of footing:

$$d_{\text{face3}} := \frac{d_{\text{edge}} + d_{\text{face2}}}{2} \quad d_{\text{face3}} = 51.23 \cdot \text{in}$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

Given  $x_{\text{cr}} := 11.49 \cdot \text{in}$

$$\text{CUBIC} = \frac{w_f \cdot x_{\text{cr}}^2}{2} - n_{\text{mod}} \cdot \left( \frac{B}{s_{\text{topm}}} \cdot A_{\text{topm}} \right) \cdot (d_{\text{face2}} - x_{\text{cr}}) \dots$$

$$+ -n_{\text{mod}} \cdot \left[ \frac{(w_f - B)}{s_{\text{topm}}} \cdot A_{\text{topm}} \right] \cdot (d_{\text{face3}} - x_{\text{cr}})$$

$$x_{\text{cr}} := \text{Find}(x_{\text{cr}}) \quad x_{\text{cr}} = 6.95 \cdot \text{in}$$

Moment of inertia of transformed section after the onset of cracking:

$$I_{\text{CR}} := \frac{w_f \cdot x_{\text{cr}}^3}{3} + n_{\text{mod}} \cdot \left( \frac{B}{s_{\text{topm}}} \cdot A_{\text{topm}} \right) \cdot (d_{\text{face2}} - x_{\text{cr}})^2 \dots$$

$$+ n_{\text{mod}} \cdot \left[ \frac{(w_f - B)}{s_{\text{topm}}} \cdot A_{\text{topm}} \right] \cdot (d_{\text{face3}} - x_{\text{cr}})^2$$

$$I_{\text{CR}} = 22.8 \cdot \text{ft}^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{\text{stmintopNorth}_{\text{qr}}} := \frac{n_{\text{mod}} \cdot M_{\text{fmintopNorth}_{\text{qr}}} \cdot (d_{\text{face2}} - x_{\text{cr}})}{I_{\text{CR}}}$$

Elastic beam theory prediction of maximum tensile stress in reinforcement:

$$\sigma_{\text{stmaxtopNorth}_{\text{qr}}} := \frac{n_{\text{mod}} \cdot M_{\text{fmaxtopNorth}_{\text{qr}}} \cdot (d_{\text{face2}} - x_{\text{cr}})}{I_{\text{CR}}}$$

Design factors:

$$C_3 = 19.60$$

$$C_4 = 6.00$$

Tensile stress range in steel:

$$\Delta\sigma_{sttopNorth_{qr}} := \max\left[1 \cdot \text{psi}, \left(\sigma_{stmaxtopNorth_{qr}} - \sigma_{stmintopNorth_{qr}}\right)\right]$$

Characteristic strength of reinforcement:

$$f_{sk} := f_{yT}$$

Material coefficient for reinforcement:

$$\gamma_s = 1.00$$

Check maximum tensile stress in steel:

$$\text{CheckStress}_{North_{max4}} := \text{if}\left(\max(\sigma_{stmaxtopNorth}) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}$$

Cycles Allowed:

$$n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{sttopNorth_{qr}}}{\text{MPa}}\right)\right)}$$

$$n_{sallowNorth_{qr}} := \text{if}\left(n_{sallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:

$$\text{Damage}_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$$

Accumulated Damage  
(Section 6, M107):

$$\text{Damage}_{stoptotalNorth} := \sum \text{Damage}_{sNorth}$$

$$\text{Damage}_{stoptotalNorth} = 0.48$$

$$\text{if}\left(\text{Damage}_{stoptotalNorth} \leq 0.33, \text{"OK"}, \text{"NG"}\right) = \text{"NG"}$$

### U. Check Fatigue Pullout Strength of Concrete (concrete only)

Minimum fatigue force due to wind:

$$P_{fat\_cone\_min\_qr} := \max \left[ 0 \text{ psi}, \left[ \frac{4 \cdot (M_{minnorth\_qr})}{\frac{N}{2} \cdot D_{avg}} - \frac{W_{mean} + W_p}{\frac{N}{2}} \right] \right]$$

Maximum fatigue force due to wind:

$$P_{fat\_cone\_max\_qr} := \left[ \frac{4 \cdot (M_{maxnorth\_qr})}{\frac{N}{2} \cdot D_{avg}} - \frac{W_{mean} + W_p}{\frac{N}{2}} \right]$$

Area of Cone

$$A_{cone} = 991.59 \cdot \text{in}^2$$

Maximum stress in concrete shear cone:

$$\sigma_{cone\_max\_qr} := \frac{P_{fat\_cone\_max\_qr}}{A_{cone}}$$

$$\max(\sigma_{cone\_max}) = 70.98 \text{ psi}$$

Minimum stress in concrete shear cone:

$$\sigma_{cone\_min\_qr} := \frac{P_{fat\_cone\_min\_qr}}{A_{cone}}$$

$$\max(\sigma_{cone\_min}) = 68.02 \text{ psi}$$

Characteristic Tensile Strength:

$$f_{tk} := 0.48 \cdot (f_{cck} \cdot \text{MPa})^{0.5} \quad f_{tk} = 2.75 \cdot \text{MPa}$$

Normalized Tensile Strength:

$$f_{tn} := f_{tk} \cdot \left[ 1 - \left( \frac{f_{tk}}{25 \cdot \text{MPa}} \right)^{0.6} \right] = 292.51 \text{ psi} \quad f_{tn} = 2.02 \cdot \text{MPa}$$

Design Tensile Strength:

$$f_{td} := \frac{f_{tn}}{\gamma_{cdnv}} \quad f_{td} = 1.49 \cdot \text{MPa}$$

Design Constants:

$$k_A := 100 \cdot \text{MPa}$$

$$d_1 := 1000 \cdot \text{mm}$$

Calibration Factor:

$$U := 0.79 \quad U \cdot f_{td} = 171 \text{ psi}$$

Additional Reduction factor:

$$\gamma_{sf} := 1.35$$

Design Shear Strength Stated in Terms of Stress:

$$v_{cd\_cone} := \frac{U \cdot f_{td} \cdot k_v}{\gamma_{sf}} \quad v_{cd\_cone} = 127 \text{ psi}$$

Cycles Allowed:

$$n_{cone\_allow\_qr} := 10 \cdot C_{1dnt} \cdot \left( \frac{1 - \frac{\sigma_{cone\_max\_qr}}{C_{5c} \cdot v_{cd\_cone}}}{1 - \frac{\sigma_{cone\_min\_qr}}{C_{5c} \cdot v_{cd\_cone}}} \right)$$

$$X_{dnv_{qr}} := \frac{C_{1dnv}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2dnv_{nr}} := \max\left[1 + 0.2 \cdot (\log(n_{cone\_allow_{nr}}) - X_{dnv_{nr}}), 1.0\right]$$

$$n_{cone\_allow_{qr}} := \text{if} \left[ \log(n_{cone\_allow_{qr}}) > X_{dnv_{qr}}, 10 \right. \\ \left. C_{1dnv} \cdot C_{2dnv_{qr}} \cdot \frac{\left( \frac{\sigma_{cone\_max_{qr}}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}}} \right)}{\left( \frac{\sigma_{cone\_min_{qr}}}{1 - \frac{\sigma_{cone\_min_{qr}}}{C_{5c} \cdot v_{cd\_cone}}} \right)}, n_{cone\_allow_{qr}} \right]$$

Damage:

$$Damage_{cone_{qr}} := \frac{N_{fat_{qr}}}{n_{cone\_allow_{qr}}}$$

Accumulated Damage  
(Section 6, M108):

$$Damage_{cone\_total} := \sum Damage_{cone}$$

$$Damage_{cone\_total} = 0.00$$

$$\text{if}(Damage_{cone\_total} < 1.0, "OK", "NG") = "OK"$$



## **Appendix B**

### **Visual and Coring Inspection Summary Report**

**(provided under separate cover)**

## **Appendix C**

**Foundation Structural Health Monitoring Summary Report**

**(provided under separate cover)**



## **Appendix D**

**Soil Thermal Resistivity Testing Results (dated December 11, 2020)**

**(provided under separate cover)**

## **Appendix E**

**Piezometer Results (dated February 5, 2021)**

**(provided under separate cover)**

## **Appendix F**

**Cyclic Degradation Geotechnical Evaluation (dated March 31, 2021)**

**(provided under separate cover)**