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September 18, 2017

Ms. Barcy F. McNeal, Secretary
Ohio Power Siting Board
Docketing Division
180 East Broad Street, 11th Floor
Columbus, OH 43215

Re: Case No. 13-197-EL-BGN, 16-1687-EL-BGA, and 17-1099-EL-BGA
Trishe Wind Ohio, LLC
Supplement to September 1, 2017 Filing Regarding Compliance with Condition 6 – Drawings for Final Design Plan

Dear Ms. McNeal:

Trishe Wind Ohio, LLC ("Applicant") is certified to construct a wind-powered electric generation facility in Paulding County, Ohio ("Project"), in accordance with the December 16, 2013 Opinion, Order, and Certificate ("Certificate") issued by the Ohio Power Siting Board.

Condition 6 of the Certificate requires the Applicant, at least 30 days prior to the preconstruction conference, to submit to staff for review and acceptance, one set of detailed engineering drawings of the final project design, including the facility, temporary and permanent access roads, and any crane routes, construction staging areas, and any other associated facilities and access points.

On September 1, 2017, Applicant filed a Notification of Compliance with Condition 6 of the Certificate. At this time, we are updating the September 1, 2017 filing to include the spread footing foundation plan and the foundation design computations, which are attached hereto.

We are available, at your convenience, to answer any questions you may have.

Respectfully submitted,

/s/ *William V. Vorys*
William V. Vorys (0093479)
Christine M.T. Pirik (0029759)
Terrence O'Donnell (0074213)
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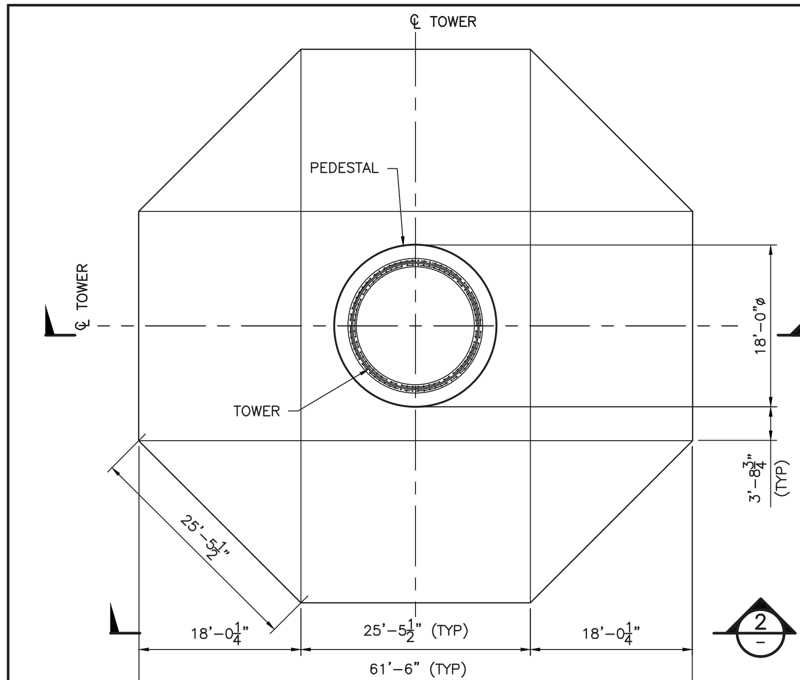
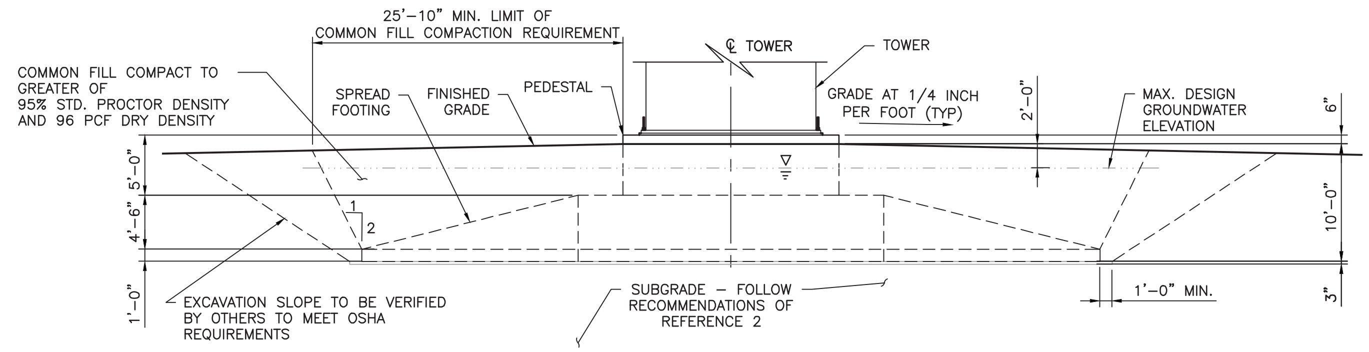
Enclosure

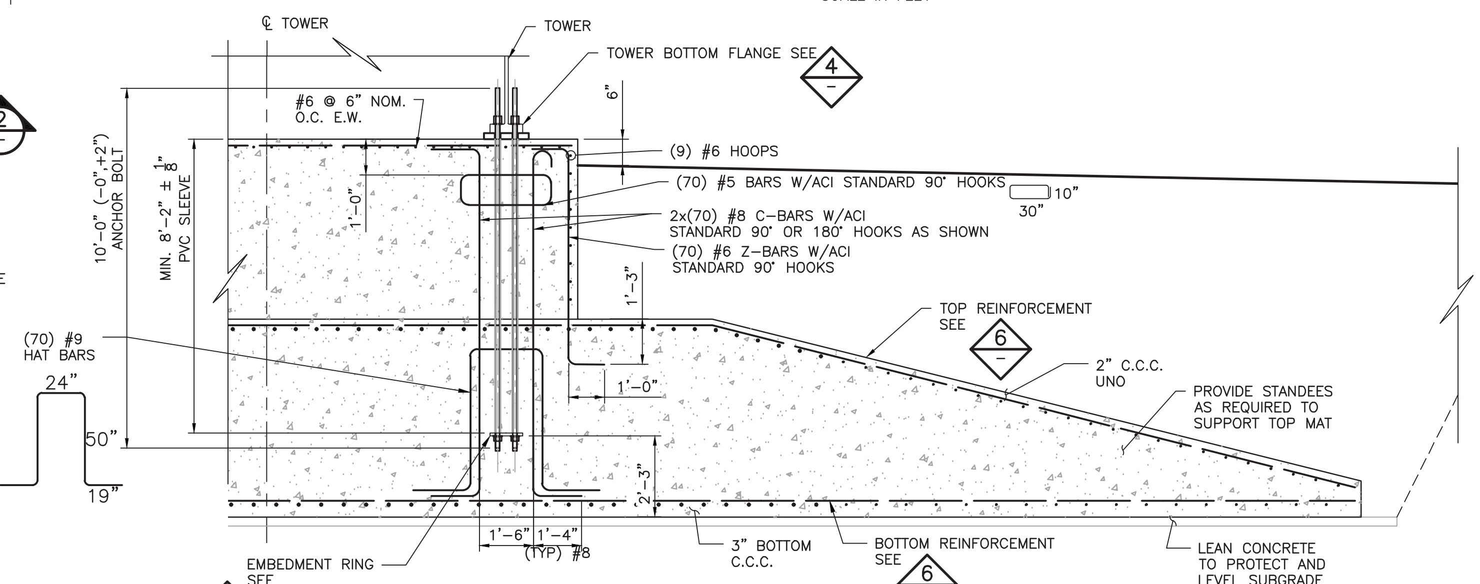
Attorneys for Trishe Wind Ohio, LLC



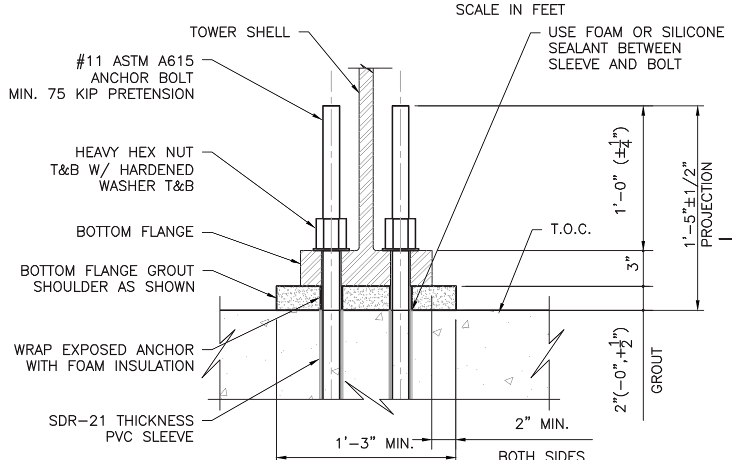
1 PLAN: TOWER FOUNDATION
3/32"=1'-0" 0 5 10 20 30 SCALE IN FEET



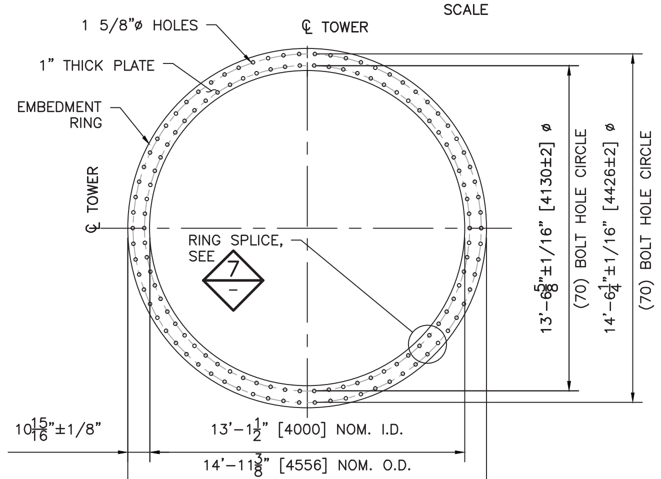
2 ELEVATION: TOWER FOUNDATION
1/8"=1'-0" 0 1 5 10 SCALE IN FEET



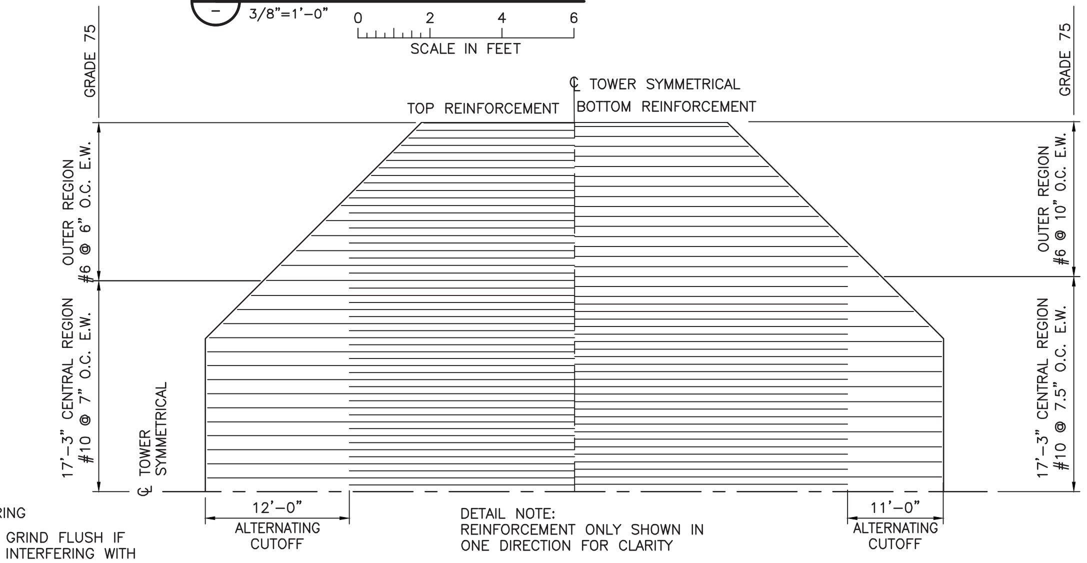
3 SECTION: TOWER FOUNDATION
3/8"=1'-0" 0 2 4 6 SCALE IN FEET



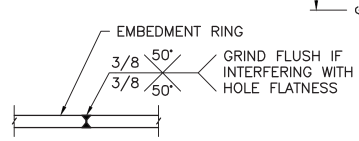
4 DETAIL: TOWER BOTTOM FLANGE
1 1/2"=1'-0" 0 3" 6" 9" 1' 2' SCALE



5 DETAIL: EMBEDMENT RING
1/4"=1'-0" 0 1 5 SCALE IN FEET



6 DETAIL: TOP & BOTTOM REINFORCEMENT
1/8"=1'-0" 0 1 5 10 SCALE IN FEET



7 DETAIL: EMBEDMENT RING SPLICE
1 1/2"=1'-0" 0 3" 6" 9" 1' 2' SCALE

ISSUE FOR CONSTRUCTION
CONFIDENTIAL

BUILDING AND DESIGN CODES:

INTERNATIONAL BUILDING CODE 2012, INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS.

BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE, ACI 318, 2011, AMERICAN CONCRETE INSTITUTE.

WIND TURBINE AND TOWER:

MANUFACTURER: GE
MODEL: 2.5-116
POWER OUTPUT: 2.5 MW
TURBINE HUB HEIGHT: 90m
ROTOR DIAMETER: 116m

DESIGN SERVICE LOADS:

UNFACTORED SERVICE LOADS DUE TO NORMAL EXTREME WIND CONDITION IEC CLASS S
(APPLY 1.4 LOAD FACTOR TO LOADS SHOWN BELOW TO OBTAIN FACTORED LOADS)
OVERTURNING MOMENT, Mxy = 45,003 kN-m
HORIZONTAL BASE SHEAR, Hxy = 512 kN
VERTICAL TOWER LOAD, Wz = 2,699 kN

UNFACTORED SERVICE LOADS DUE TO ABNORMAL EXTREME WIND CONDITION IEC CLASS S
(APPLY 1.14 LOAD FACTOR TO LOADS SHOWN BELOW TO OBTAIN FACTORED LOADS)
OVERTURNING MOMENT, Mxy = 55,693 kN-m
HORIZONTAL BASE SHEAR, Hxy = 692 kN
VERTICAL TOWER LOAD, Wz = 2,625 kN

TOWER MISALIGNMENT LOADING, Mxy = 1,335 kN-m

FOUNDATION DESIGN DATA:

FATIGUE LIFE: 20 YEARS BASED ON FATIGUE LOADING PROVIDED IN REFERENCE 1

REFERENCE DOCUMENTS:

1. GE RENEWABLE ENERGY A/S, "FOUNDATION LOAD SPECIFICATION FOR WIND TURBINE GENERATOR SYSTEMS, STARWOOD OHIO/USA (GE PROJECT 1022819) 2.5-116 60 Hz, 90m HUB HEIGHT, COLD WEATHER EXTREME, IEC CLASS S", r01 DATED 2017-09-05.

2. BARR ENGINEERING COMPANY, "GEOTECHNICAL ENGINEERING REPORT, NORTHWEST OHIO WIND PROJECT, PAULDING COUNTY, OHIO", PREPARED FOR STARWOOD ENERGY GROUP GLOBAL, DECEMBER 2014

3. BARR ENGINEERING COMPANY, "FOUNDATION DESIGN CALCULATIONS, GE 2.5-116 90m HUB HEIGHT, NORTHWEST OHIO WIND PROJECT, PAULDING COUNTY, OHIO", PREPARED FOR WHITE CONSTRUCTION INC., DATED SEPTMEBER 2017.

MIN. 28-DAY COMPRESSIVE STRENGTH CONCRETE
5,000 PSI

MIN. YIELD POINT STRENGTH OF REINFORCING BAR:
TOP AND BOTTOM REINFORCEMENT: **75 KSI**
ALL OTHER REINFORCEMENT: **60 KSI**

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

MIN. 28-DAY COMPRESSIVE STRENGTH OF NON-SHRINK GROUT:
9,500 PSI

MIN. YIELD POINT STRENGTH OF EMBEDMENT PLATE:
36 KSI

ESTIMATED CONCRETE VOLUME:
FOOTING: **410 CUBIC YARDS**
PEDESTAL: **47 CUBIC YARDS**

ESTIMATED WEIGHT OF STEEL REINFORCING:
REINFORCEMENT:
GRADE 75: **29.1 TONS**
GRADE 60: **5.7 TONS**

COARSE AGGREGATE GRADATION:
ASTM C33 (SIZE NUMBER 6 OR 67) WITH A MINIMUM OF 2% RETAINED ON THE 3/4-INCH SIEVE.

ABBREVIATIONS:

B.O.	BOTTOM OF	O.C.	ON CENTER
C.C.C	CLEAR CONCRETE COVER	O.D.	OUTSIDE DIAMETER
℄	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	MAX.	MAXIMUM
NOM.	NOMINAL	W/	WITH
ø	DIAMETER		

STATE OF OHIO
MATTHEW B. JOHNSON
E-74181
09/15/17
REGISTERED PROFESSIONAL ENGINEER

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
A	LJH	CMM3	MBJ	7/21/17	ISSUED FOR PROCUREMENT
B	LJH	CMM3	MBJ	7/31/17	REVISED ISSUE FOR PROCUREMENT
C	LJH	CMM3	MBJ	8/10/2017	ISSUE FOR IE REVIEW
0	RRR2	CMM3	MBJ	9/15/2017	ISSUED FOR CONSTRUCTION

RELEASED TO/FOR	A	B	C	0	1	2	3
CLIENT	07/21/17	07/31/17	08/10/17				
BID							
CONSTRUCTION				09/15/17			
	DATE RELEASED						

BARR
Corporate Headquarters: Minneapolis, Minnesota Ph: 1-800-632-2277

Project Office:
BARR ENGINEERING CO.
4300 MARKETPOINTE DRIVE
Suite 200
MINNEAPOLIS, MN 55435
Ph: 1-800-632-2277
Fax: (952) 832-2601
www.barr.com

Scale	AS SHOWN
Date	3/17/2017
Drawn	LJH
Checked	CMM3
Designed	LJH
Approved	MBJ

WHITE CONSTRUCTION
CLINTON, INDIANA

NORTHWEST OHIO WIND PROJECT
PAULDING COUNTY, OHIO

SPREAD FOOTING FOUNDATION
PLAN, ELEVATION, SECTION & DETAILS

BARR PROJECT No. 35631001.02
CLIENT PROJECT No.
DWG. No. S-01
REV. No. 0

1.0 GENERAL REQUIREMENTS AND SUBMITTALS

A. GENERAL
1. THE REQUIREMENTS SPECIFIED HEREIN APPLY TO THE FOLLOWING DRAWINGS:
   a. NORTHWEST OHIO WIND PROJECT, DRAWING S-01, SPREAD FOOTING FOUNDATION

B. SUBMITTALS
1. SUBMITTALS SHALL BE MADE A MINIMUM OF ONE WEEK PRIOR TO INCORPORATION INTO THE WORK. THE FOUNDATION ENGINEER (BARR) WILL REVIEW SUBMITTALS, INCLUDING THE TESTING AND INSPECTION RECORDS TO CHECK CONFORMANCE WITH THE DRAWINGS AND SPECIFICATIONS. THE REVIEW DOES NOT RELIEVE THE CONTRACTOR FROM RESPONSIBILITY FOR ERRORS IN CONSTRUCTION OF THE WORK DUE TO ERRORS CONTAINED IN THOSE DOCUMENTS.
2. SUBMIT ONE ELECTRONIC COPY OF THE SUBMITTALS SPECIFIED TO THE FOUNDATION ENGINEER AT THE FOLLOWING: BARR ENGINEERING COMPANY

ATTN: MR. CHUCK BEAUZAY [CBEAUZAY@BARR.COM]

3. SUBMIT A LIST OF THE TESTING COMPANIES THAT WILL BE UTILIZED ON THE PROJECT FOR PERFORMANCE OF TESTS SPECIFIED.
4. SUBMIT NAME AND QUALIFICATIONS OF THE GEOTECHNICAL ENGINEER.
5. SUBMIT INFORMATION (TESTING RESULTS, PRODUCT DATA, CONSTRUCTION DETAILS, ETC.) AS LISTED IN THE FOLLOWING SECTIONS: 2.B, 3.B, 4.B, 5.B, AND 6.B.

2.0 EXCAVATION, SUBGRADE PREPARATION, BACKFILL, & COMPACTION

A. GENERAL
1. COORDINATE THE EXCAVATION, SUBGRADE PREPARATION, BACKFILL, COMPACTION, AND GRADING ACTIVITIES WITH THE REFERENCED GEOTECHNICAL DOCUMENTS ON DRAWING S-01.

B. SUBMITTALS
1. SUBMIT GROUNDWATER AND SURFACE WATER CONTROL PLAN.
2. SUBMIT SUBGRADE STRENGTH AND UNIFORMITY VERIFICATION METHOD.
3. SUBMIT SUBGRADE INSPECTION REPORT FOR EACH FOUNDATION COMPLETED BY A GEOTECHNICAL ENGINEER.
4. SUBMIT GRAIN SIZE ANALYSIS PER ASTM D422, NATURAL MOISTURE CONTENT PER ASTM D2216, AND STANDARD PROCTOR MAXIMUM DRY DENSITY PER ASTM D698 FOR COMMON FILL SOIL MATERIALS.
5. SUBMIT COMPACTION TEST RESULTS FOR FILL PLACED OVER THE FOUNDATION INDICATING LOCATION OF TEST, DRY DENSITY, AND MOISTURE CONTENT OF PLACED FILL.

C. PRODUCTS
1. **LEAN CONCRETE:** CONTAINING ASTM C150, TYPE I OR ASTM C1157, TYPE GU CEMENT. COMPRESSIVE STRENGTH AND THICKNESS SHALL BE SUFFICIENT TO SUPPORT REINFORCING STEEL AND ANCHOR BOLT CAGE DURING CONSTRUCTION.
2. **COMMON FILL:** SHALL CONSIST OF SUITABLE UNFROZEN MATERIALS EXCAVATED FROM THE FOUNDATION SITE OR IMPORTED AS NECESSARY. ADDITIONAL CRUSHING AND SCREENING MAY BE REQUIRED TO PROCESS THE MATERIAL TO THE SPECIFIED REQUIREMENTS BELOW.
   a. MATERIALS BACKFILLED WITHIN 1 FOOT OF ANY CONCRETE SHALL BE FINE, WELL GRADED MATERIAL WITH PARTICLE SIZE NO GREATER THAN 3 INCHES.
   b. MATERIALS BACKFILLED BEYOND 1 FOOT OF ANY CONCRETE MAY CONSIST OF ALL OTHER EXCAVATED MATERIALS PROVIDED THEY MEET THE DENSITY REQUIREMENTS AND CAN BE PLACED USING METHODS THAT WILL PREVENT VOIDS FROM OCCURRING.
3. **ENGINEERED FILL:** CONTACT FOUNDATION ENGINEER IF NEEDED BASED ON SUBGRADE INSPECTION.

D. EXECUTION
1. CONFIRM LOCATION OF TURBINE COORDINATES IN THE REFERENCED GEOTECHNICAL DOCUMENT ON DRAWING S-01. IF TURBINE COORDINATES ARE OFFSET BY MORE THAN 50 FEET, OBTAIN WRITTEN INSTRUCTIONS FROM THE FOUNDATION ENGINEER AS TO THE MEANS OF ADDITIONAL INVESTIGATION TO BE UNDERTAKEN. OBTAIN WRITTEN CONFIRMATION FROM THE GEOTECHNICAL ENGINEER THAT THE SPECIFIED INVESTIGATION WAS COMPLETED.
2. REMOVE TOPSOIL FROM THE PLAN AREA AND STORE IN AN OWNER DESIGNATED AREA. THE TOPSOIL SHALL BE USED FOR SITE RESTORATION.
3. EXCAVATE SOILS OR ROCK TO THE LIMITS INDICATED ON DRAWING S-01 USING TECHNIQUES THAT WILL MINIMIZE DISTURBANCE TO THE SUBGRADE. CONTRACTOR SHALL BE RESPONSIBLE FOR CONTROL OF SURFACE WATER AND/OR GROUNDWATER FLOWS INTO THE EXCAVATION.
4. COMPACTION OF IN-SITU SOILS IS NOT REQUIRED WHERE CLAYEY SOILS ARE PRESENT AT THE EXCAVATION BASE. AT TURBINE SITES WHERE SANDS ARE ENCOUTERED AT THE FOUNDATION BASE ELEVATION, PERFORM SURFACE COMPACTION WITH A MINIMUM OF ONE PASS THROUGHOUT THE EXCAVATION BASE.
5. IF, IN THE COURSE OF EXCAVATING THE FOUNDATION, THE BASE OF THE EXCAVATION BECOMES RUTTED, DAMAGED OR IS OTHERWISE DETERMINED TO BE OF INADEQUATE CHARACTER, PERFORM THE FOLLOWING ACTIONS:
   a. SILTS OR CLAYS: SUBCUT THE EXCAVATION A MINIMUM OF 6 INCHES BEYOND THE DEPTH OF THE INADEQUATE SOILS AND REPLACE WITH LEAN CONCRETE OR ENGINEERED FILL. CONTACT FOUNDATION ENGINEER FOR ENGINEERED FILL REQUIREMENTS.
   b. GRANULAR SOILS: LEVEL AND SURFACE COMPACT THE EXCAVATION BASE SURFACE COMPACTION OF THIS MATERIAL TO ACHIEVE AT LEAST 98 PERCENT OF THE LABORATORY MAXIMUM DRY DENSITY MEASURED ACCORDING TO THE STANDARD PROCTOR TEST METHOD.
6. PRIOR TO PLACING PROTECTIVE LEAN CONCRETE SURFACE, HAVE A PROFESSIONAL GEOTECHNICAL ENGINEER (OR A PERSON UNDER THE GEOTECHNICAL ENGINEER'S DIRECT SUPERVISION) INSPECT THE SUBGRADE CONDITIONS AND RECORD THE SOIL TYPE ENCOUNTERED, GROUNDWATER CONDITIONS, OR OTHER SUBSURFACE CONDITIONS. A SUBGRADE INSPECTION REPORT SHALL BE PREPARED AND SUBMITTED FOR EACH FOUNDATION THAT INCLUDES THE FOLLOWING:
   a. VERIFICATION THAT OBSERVATIONS TAKEN ARE CONSISTENT WITH THE OBSERVATIONS CONTAINED IN THE REFERENCED GEOTECHNICAL DOCUMENT ON DRAWING S-01.
   b. VERIFICATION THAT SUBGRADE STRENGTH AND UNIFORMITY ARE ADEQUATE (SUBMIT FOR REVIEW THE METHODS TO BE USED TO VERIFY THE SUBGRADE STRENGTH AND UNIFORMITY).
   c. PHOTOS OF PREPARED SUBGRADE.

   IF SOIL CONDITIONS ARE ENCOUNTERED THAT ARE NOT CONSISTENT WITH THE REFERENCED GEOTECHNICAL DOCUMENTS (E.G. HALF SOILS AND HALF ROCK) OR IF SUBGRADE UNIFORMITY OR STRENGTH IS INSUFFICIENT, 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 CORRECTIVE ACTIONS WERE COMPLETED.
7. FOR PROTECTION OF THE SUBGRADE AND ESTABLISHMENT OF A WORKING SURFACE, PLACE LEAN CONCRETE FILL AS INDICATED ON DRAWING S-01. IT IS RECOMMENDED THAT THE LEAN CONCRETE FILL BE PLACED AS LEVEL AS PRACTICAL TO FACILITATE PLACEMENT OF THE REINFORCING STEEL AND EMBEDMENT RING.
8. BACKFILL AND COMPACTION: PLACE AND COMPACT COMMON FILL MATERIALS TO THE LIMITS, DEPTH AND DRY DENSITY INDICATED ON DRAWING S-01. IN ADDITION TO THE DRY DENSITY REQUIREMENT, BACKFILL MUST BE COMPACTED TO A MINIMUM OF 95% STANDARD PROCTOR. PLACE FILL IN MAXIMUM LOOSE LIFTS OF 12 INCHES OR LESS TO ACHIEVE THE SPECIFIED DENSITY. ADDITIONAL DRYING OF BACKFILL MATERIAL MAY BE NECESSARY TO ACHIEVE THESE SPECIFICATIONS. BACKFILL MAY BE PLACED WHEN THE FOOTING AND PEDESTAL HAVE REACHED 2,000 PSI.
9. GRADE THE SITE IN ACCORDANCE WITH DRAWING S-01 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
1. FOR EVERY 2500 CUBIC YARDS OF PLACED COMMON FILL, OBTAIN SAMPLES OF COMMON FILL MATERIALS AND PERFORM AND SUBMIT GRAIN SIZE ANALYSIS PER ASTM D422, MOISTURE CONTENT PER ASTM D2216, AND STANDARD PROCTOR MAXIMUM DRY DENSITY PER ASTM D698.
2. FOR ALL PLACED AND COMPACTED COMMON FILLS AROUND THE FOUNDATION, PERFORM AND SUBMIT ONE DENSITY TEST PER LIFT INDICATING TEST LOCATION, DRY DENSITY AND MOISTURE CONTENT PER ASTM D6938.
3. PROVIDE A SUBGRADE INSPECTION REPORT TO BE COMPLETED BY A GEOTECHNICAL ENGINEER FOR EACH FOUNDATION.

3.0 CAST-IN-PLACE CONCRETE AND STEEL REINFORCING

A. GENERAL
1. CONCRETE WORK SHALL BE IN COMPLIANCE WITH THE FOLLOWING CODES AND SPECIFICATIONS:
   a. ACI 301, STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE.
   b. ACI 308, STANDARD SPECIFICATION FOR CURING CONCRETE.
   c. ACI 318 (CURRENT EDITION), BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE.
   d. ASTM C94, STANDARD SPECIFICATION FOR READY-MIX CONCRETE.
   e. ASTM C172, STANDARD PRACTICE FOR SAMPLING FRESHLY MIXED CONCRETE.
2. CONCRETE SHALL MEET THE REQUIREMENTS OF ACI 318, TABLES 19.3.1.1 AND 19.3.2.1 FOR EXPOSURE CLASSES 'F2', 'S0', 'W0', AND 'C1'.

B. SUBMITTALS
1. FOR EACH CONCRETE TYPE USED, SUBMIT FOR APPROVAL A MIX DESIGN CERTIFIED BY A PROFESSIONAL ENGINEER (LICENSED IN OHIO) AND MEETING THE MINIMUM SPECIFIED REQUIREMENTS. CONCRETE MIX SHALL BE PROPORTIONED ACCORDING TO THE REQUIREMENTS OF ACI 318, CHAPTER 5 ON THE BASIS OF FIELD DATA OR TRIAL MIXTURES.
2. SUBMIT PRODUCT DATA FOR ADMIXTURES, POZZOLAN, AND CEMENT USED ON THE PROJECT.
3. SUBMIT GRADATION, SOURCE, AND TYPE OF COARSE AND FINE AGGREGATE MEETING THE REQUIREMENTS OF ASTM C33.
4. SUBMIT REINFORCING FABRICATION AND PLACEMENT SHOP DRAWINGS.
5. SUBMIT MILL REPORTS OF REINFORCING STEEL, CONFIRMING THE GRADE AND STRENGTH OF REINFORCING STEEL PROVIDED ON THE PROJECT.
6. SUBMIT QUALITY CONTROL FIELD TESTS OF AIR CONTENT, SLUMP, AIR TEMPERATURE, AND CONCRETE TEMPERATURE.
7. SUBMIT CONCRETE CYLINDER STRENGTH TEST RESULTS.
8. SUBMIT A PLAN FOR HOT AND COLD WEATHER PROTECTION OF CONCRETE IN ACCORDANCE WITH ACI 305-306.
9. SUBMIT A PLAN FOR CONCRETE CURING IN ACCORDANCE WITH ACI 308.
10. ASR REQUIREMENTS: IF AGGREGATES CONTAIN POTENTIALLY REACTIVE MATERIALS (AS DETERMINED BY ONE OF THE TEST METHODS OUTLINED IN ASTM C33, APPENDIX X1), SUBMIT TEST RESULTS INDICATING THE POTENTIAL REACTIVITY, SUCH AS THE RESULTS OF TESTING TO ASTM C295, C289, C1293, OR C1260. IF THESE TEST RESULTS INDICATE THE AGGREGATES ARE REACTIVE, SUBMIT AN ASR MITIGATION PLAN, INCLUDING VERIFICATION THAT THE PROPOSED MEASURES WILL SUFFICIENTLY LIMIT ASR TO PREVENT EXCESSIVE EXPANSION. THIS VERIFICATION SHALL CONSIST OF THE RESULTS OF TESTS PERFORMED ACCORDING TO ASTM C1567, AASHTO T303, OR ASTM C1293.
11. SUBMIT FOR APPROVAL A MASS CONCRETE PLACEMENT AND TEMPERATURE CONTROL PLAN MEETING THE REQUIREMENTS OF ACI 301 CHAPTER 8 AND ACI 207.1R.

C. PRODUCTS
1. **REINFORCING BARS:** TO ASTM A615, GRADE 60 OR GRADE 75 AS NOTED ON DRAWING S-01, DEFORMED, UNCOATED.
2. **CEMENT:** TO ASTM C150, TYPE I, OR ASTM C1157, TYPE GU.
3. **FLY ASH:** TO ASTM C618, CLASS C OR F (IF SPECIFIED).
4. **MINIMUM CEMENTITIOUS CONTENT:** IN ACCORDANCE WITH APPROVED MIX DESIGN.
5. **COARSE AND FINE AGGREGATES:** TO ASTM C33, GRADATION IN ACCORDANCE WITH SPECIFICATIONS AND APPROVED MIX DESIGN. NOMINAL MAXIMUM AGGREGATE SIZE SHALL BE AS SHOWN ON DRAWING S-01. ALL AGGREGATES MUST BE NON-REACTIVE WITH CEMENT TO PREVENT ASR.
6. **AIR ADMIXTURE AND CONTENT:** TO ASTM C260, 6% FOR PEDESTAL ONLY, NO AIR CONTENT REQUIREMENT FOR FOOTING.
7. **OTHER ADMIXTURES:** CHLORIDE FREE WATER REDUCING ADMIXTURE AND SUPERPLASTICIZER AS REQUIRED.
8. **MAXIMUM WATER CEMENT RATIO:** 0.45.
9. **28 DAY COMPRESSIVE STRENGTH:** 5,000 PSI.
10. **SLUMP:** IN ACCORDANCE WITH APPROVED MIX DESIGN AT THE POINT OF DEPOSITION WITH THE ADDITION OF ADMIXTURES.
11. **CONCRETE UNIT WEIGHT:** 145 PCF (MINIMUM) TO ASTM C138.

D. EXECUTION
1. PLACE CONCRETE AND REINFORCING AS SHOWN AND IN ACCORDANCE WITH THE FOLLOWING TOLERANCES:
   a. REINFORCING PLAN SPACING: PLUS OR MINUS 2 INCHES.
   b. REINFORCING VERTICAL SPACING: PLUS OR MINUS 1 INCH.
   c. FOOTING CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 3 INCHES.
   d. PEDESTAL CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 2 INCHES.
   e. FOOTING PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 3 INCHES.
   f. FOOTING THICKNESS: MINUS 0 INCHES, PLUS 3 INCHES.
   g. PEDESTAL PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 2 INCHES.
   h. PEDESTAL HEIGHT: MINUS 1 INCH, PLUS 0 INCHES.
   i. PEDESTAL CENTERED TO WITHIN 2 INCHES RELATIVE TO FOOTING.
   j. CONCRETE AIR CONTENT: +/- 1.5%
2. PROVIDE NECESSARY TIES, CHAIRS, AND STANDEES TO SECURE AND SUPPORT REBAR AND PREVENT PERMANENT DISPLACEMENT OR MOVEMENT OF THE BARS GREATER THAN 1 INCH DURING PLACEMENT OF CONCRETE. REBAR THAT DEFLECTS BUT RETURNS TO ITS ORIGINAL POSITION IS ACCEPTABLE.
3. REINFORCEMENT SHALL BE FREE OF LOOSE RUST, MILL SCALE, EARTH, ICE, CONCRETE, OR OTHER MATERIALS WHICH COULD PREVENT BONDING TO NEW CONCRETE.
4. SET FORMWORK PER ACI 347 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.
5. PLACE CONCRETE IN ACCORDANCE WITH ACI 318. PLACE SUCCESSIVE LIFTS OF CONCRETE AS QUICKLY AS POSSIBLE TO ENSURE PROPER AMALGAMATION OF CONCRETE BETWEEN SUCCESSIVE LIFTS.
6. CONSOLIDATE CONCRETE IN ACCORDANCE WITH ACI 318 PREVENTING THE FORMATION OF JOINTS, VOIDS, HONEYCOMBING OR SEGREGATION OF AGGREGATE.
7. ROUGH FINISH TOP OF CONCRETE FOOTING USING A ROLLER SCREED.
8. PRIOR TO PLACING PEDESTAL CONCRETE, CLEAN CONCRETE SURFACE WITH AIR OR WATER TO REMOVE DEBRIS AND OTHER LOOSE MATERIAL FROM TOP OF FOOTING.
9. TROWEL AND BROOM FINISH TOP OF PEDESTAL.
10. CURE CONCRETE FOOTING AND PEDESTAL IN ACCORDANCE WITH ACI 318 AND 308. IF A CURING MEMBRANE IS USED, APPLY CURING MEMBRANE AS SOON AS BLEEDING HAS STOPPED AND FREE WATER HAS DISAPPEARED FROM THE SURFACE.
11. 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.
12. ALL HOOKS SHOWN ON REBAR SHALL BE STANDARD HOOKS (UNO).
13. ANY SHRINKAGE CRACKS IN EXCESS OF 0.012 INCHES (0.3mm) IN WIDTH SHALL BE SEALED WITH AN ENGINEER APPROVED PRODUCT.
14. JOBSITE ADDITION OF WATER TO AIR ENTRAINED CONCRETE IS PROHIBITED.
15. MONITOR MASS CONCRETE TEMPERATURES IN ACCORDANCE WITH THE MASS CONCRETE TEMPERATURE CONTROL PLAN.

E. TESTING AND INSPECTION
1. FOR EACH FOOTING PLACED, CAST A MINIMUM OF (2) 6-INCH OR (3) 4-INCH DIAMETER CONCRETE CYLINDERS PER ASTM C31 FOR EVERY 150 CUBIC YARDS, OR FRACTION THEREOF, OF CONCRETE PLACED FOR LABORATORY STRENGTH TESTING PER ASTM C39. PERFORM ONE "STRENGTH TEST" AT 28 DAYS FOR EVERY 150 CUBIC YARDS, OR FRACTION THEREOF, OF CONCRETE PLACED ("STRENGTH TEST" = AVERAGE OF (2) 6-INCH OR (3) 4-INCH CYLINDER BREAKS). FOR EACH FOOTING PLACED, CAST (2) 6-INCH OR (3) 4-INCH ADDITIONAL CONCRETE CYLINDERS PER ASTM C31, AND IF NECESSARY PERFORM ONE "STRENGTH TEST" PER ASTM C39 AT 56 DAYS. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
2. FOR EACH PEDESTAL, CAST A MINIMUM OF (4) 6-INCH OR (6) 4-INCH DIAMETER CONCRETE CYLINDERS PER ASTM C31 FOR LABORATORY STRENGTH TESTING PER ASTM C39. PERFORM ONE "STRENGTH TEST" AT 28 DAYS ("STRENGTH TEST" = AVERAGE OF (2) 6-INCH OR (3) 4-INCH CYLINDER BREAKS) AND IF NECESSARY ONE AT 56 DAYS. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
3. PERFORM A MINIMUM OF ONE AIR TEST PER ASTM C231 AND A MINIMUM OF ONE SLUMP TEST PER ASTM C143 PER SET OF CYLINDERS CAST. RECORD AMBIENT AIR TEMPERATURE AND CONCRETE TEMPERATURE PER ASTM C1064.
4. PERFORM TESTING AND INSPECTION REQUIRED BY THE MASS CONCRETE TEMPERATURE CONTROL PLAN.

4.0 ANCHOR BOLTS AND EMBEDMENT RING

A. GENERAL
1. PRODUCTS, SUBMITTALS, EXECUTION, AND TESTING ARE SPECIFIED TO PROVIDE DURABLE ANCHOR BOLTS AND EMBEDMENT PLATES.

B. SUBMITTALS
1. SUBMIT PRODUCT DATA AND SHOP DRAWING FOR ANCHORS AND HARDWARE.
2. SUBMIT A 12-INCH LONG PRODUCT SAMPLE OF THE ANCHOR COMPLETE WITH WASHER AND NUT.
3. SUBMIT MILL CERTIFICATES FOR ANCHORS INDICATING YIELD AND TENSILE STRENGTH OF ANCHORS.
4. SUBMIT MILL CERTIFICATES FOR THE EMBEDMENT RING INDICATING THAT THE MATERIAL MEETS THE MINIMUM STRENGTH REQUIREMENTS.
5. SUBMIT LABORATORY TENSION TESTS OF ANCHOR COMPLETE WITH THREADS.
6. SUBMIT A TENSIONING CALIBRATION PROCEDURE FOR REVIEW, INCLUDING VERIFICATION THAT THE EQUIPMENT PROVIDED AND TENSIONING METHODS USED ARE DELIVERING THE NECESSARY LOCK OFF LOAD.
7. SUBMIT A TENSIONING PROCEDURE FOR REVIEW.
8. SUBMIT A TENSION TESTING PROCEDURE FOR REVIEW.
9. SUBMIT TENSION TEST DATA FOR ANCHOR BOLTS THAT ARE TESTED INDICATING BOLT LOCATION AND TENSION VALUE.
10. SUBMIT EMBEDMENT RING AND TEMPLATE RING SHOP DRAWINGS.

C. PRODUCTS
1. **ANCHOR BOLTS:** #11 SIZE WITH MATERIAL TO ASTM A615 GRADE 75, WITH COLD ROLLED THREADS, A MINIMUM YIELD STRENGTH OF 75 KSI, A MINIMUM TENSILE STRENGTH OF 100 KSI, A MAXIMUM THREAD DIAMETER OF 1.50 INCHES, AND A MINIMUM NET AREA OF 1.56 SQUARE INCHES.
2. **ANCHOR BOLT SLEEVES:** TO ANCHOR BOLT MANUFACTURER'S REQUIREMENTS.
3. **EMBEDMENT RING:** TO ASTM A36, PLAIN FINISH. NEW MATERIAL (NO REUSED TEMPLATES).
4. **HEAVY HEX NUTS:** TO ANCHOR BOLT MANUFACTURER'S SPECIFICATIONS. NUTS SHALL BE CAPABLE OF DEVELOPING THE MINIMUM TENSILE STRENGTH OF THE ANCHOR.
5. **HARDENED STEEL WASHERS:** TO ASTM F436, PLAIN FINISH.

D. EXECUTION
1. THE FOLLOWING TOLERANCES SHALL BE ADHERED TO FOR PLACEMENT OF ANCHOR BOLTS:
   a. ANCHOR BOLT PLAN LOCATION - PLUS OR MINUS 1/16 INCH.
   b. ANCHOR BOLT PLUMBNESS - LESS THAN 1/4 DEGREE.
   c. TEMPLATE AND EMBEDMENT RING PLAN DIMENSION - PLUS OR MINUS 1/16 INCH.
   d. EMBEDMENT RING LEVEL - PLUS OR MINUS 1/4 INCH.
   e. EMBEDMENT RING ELEVATION - PLUS OR MINUS 1/2 INCH.
2. THE BOTTOM OF THE ANCHOR BOLT SHALL EXTEND BEYOND THE BOTTOM NUT BY A MINIMUM OF 1/2 INCH.
3. USE A TEMPLATE RING TO SET ANCHOR BOLT PLUMBNESS AND POSITION. ENSURE THE TEMPLATE RING IS SET IN ACCORDANCE WITH THE SPECIFIED CONSTRUCTION TOLERANCES.
4. 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.
5. 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.
6. AFTER SETTING AND GROUTING OF THE LOWER TOWER SECTION(S) AND AFTER THE CONCRETE AND GROUT HAS ACHIEVED THE REQUIRED STRENGTH GIVEN IN SECTION 7.0, USE AN APPROVED TENSIONING PROCEDURE TO APPLY A LOCK-OFF FORCE TO EACH ANCHOR BOLT WHICH IS NO GREATER THAN 8 KIPS MORE THAN THE SPECIFIED TENSION FORCE. 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 TO ENSURE REQUIRED TENSIONS ARE ACHIEVED.

E. TESTING AND INSPECTION
1. SUBMIT 3 LABORATORY TENSION TESTS FOR ANCHOR BOLTS FOR EACH HEAT NUMBER FURNISHED, COMPLETE WITH THREADS, PERFORMED BY AN INDEPENDENT TESTING LABORATORY. PERFORM TEST IN ACCORDANCE WITH ASTM A370, AND REPORT YIELD STRESS AND TENSILE STRESS.
2. AFTER ALL BOLTS HAVE BEEN TENSIONED, A MINIMUM OF 10% OF THE TOTAL BOLTS INSTALLED PER FOUNDATION SHALL BE RANDOMLY TESTED TO VERIFY THAT THE SPECIFIED 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.

5.0 TOWER BASE GROUT

A. GENERAL
1. COORDINATE GROUTING PROCEDURES WITH THE REQUIREMENTS OF THE TOWER MANUFACTURER.

B. SUBMITTALS
1. SUBMIT MANUFACTURER'S GROUT PRODUCT DATA AND MANUFACTURER'S APPROVED MIXING, PLACING AND CURING INSTRUCTIONS FOR GROUT TO BE PLACED.
2. SUBMIT GROUT CUBE STRENGTH TEST RESULTS.
3. SUBMIT CONTRACTOR'S TOWER BASE SETTING/GROUTING PLAN.

C. PRODUCTS
1. **EPOXY NON-SHRINK GROUT:** PREPACKAGED EPOXY GROUT WITH A MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS ACCORDING TO ASTM C579 AS SHOWN ON DRAWING S-01 AND A MAXIMUM COEFFICIENT OF THERMAL EXPANSION OF 30 X 10-6 IN/IN/°F IN ACCORDANCE WITH ASTM C531.
2. **CEMENTITIOUS NON-SHRINK GROUT:** PREPACKAGED GROUT CONFORMING TO ASTM C1107, WITH A MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS ACCORDING TO ASTM C109, AS SHOWN ON DRAWING S-01.

D. EXECUTION
1. MIX, PLACE, AND CURE GROUT IN ACCORDANCE WITH APPROVED MANUFACTURER'S INSTRUCTIONS.
2. FOR CEMENT GROUTS, PROVIDE GROUT SHOULDERS IN ACCORDANCE WITH DRAWING DETAILS. DO NOT ALLOW GROUT TO BE PLACED AGAINST THE SIDE OF THE TOWER FLANGE.
3. FOR EPOXY GROUTS, POUR GROUT ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS. IF GROUT IS PLACED UP THE SIDE OF THE TOWER FLANGE, PROVIDE A 1/4 INCH EXPANSION JOINT BETWEEN THE TOWER FLANGE AND THE GROUT, AND SEAL EXPANSION JOINT WITH AN APPROVED SEALANT.
4. ONCE GROUT CUBES ARE MOLDED IN THE FIELD THEY SHALL REMAIN UNDISTURBED AND PROTECTED FROM EXTREMES IN TEMPERATURE AND VIBRATION AT THE PROJECT SITE FOR AT LEAST 18 HOURS.

E. TESTING
1. CAST MINIMUM OF 9 GROUT CUBES FOR EACH FOUNDATION.
2. PERFORM TWO LABORATORY "GROUT STRENGTH TESTS" PER ASTM C109 AT 28 DAYS ("GROUT STRENGTH TEST" = AVERAGE OF THREE CUBE BREAKS) AND IF NECESSARY ONE AT A LATER DATE. CAST ADDITIONAL GROUT CUBES AS REQUIRED TO DETERMINE STRENGTH AT OTHER TIMES.

6.0 MISCELLANEOUS CONCRETE EMBEDMENTS

A. GENERAL
1. COORDINATE THE LOCATION AND PLACEMENT OF GROUNDING GRIDS, CONTROL CONDUIT AND ELECTRICAL CONDUIT.

B. SUBMITTALS
1. 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
1. NO ITEMS.

D. EXECUTION
1. VERIFY THE LOCATION OF MISCELLANEOUS CONCRETE EMBEDMENTS AND CONDUIT SO AS NOT TO INTERFERE WITH THE FOUNDATION'S STRUCTURAL REINFORCING STEEL.
2. ENSURE THAT MISCELLANEOUS EMBEDMENTS ARE PROPERLY SECURED TO PREVENT MOVEMENT DURING CONCRETE PLACEMENT.
3. TOP OF CONDUIT MUST BE A MINIMUM OF 24 INCHES BELOW TOP OF PEDESTAL.

7.0 TOWER ERECTION AND ANCHOR TENSIONING REQUIREMENTS

A. GENERAL
1. TOWER SECTIONS MAY BE ERECTED, LEVELED AND GROUTED IN ACCORDANCE WITH SUBMITTAL 5.B.3 ABOVE.
2. ANCHORS MAY BE TENSIONED WHEN:
   a. THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED 5,000 PSI.
   b. THE GROUT STRENGTH HAS REACHED 5,000 PSI.
3. THE NACELLE AND BLADES MAY BE ERECTED WHEN:
   a. THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
   b. THE GROUT STRENGTH HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
   c. UPON COMPLETION OF THE ANCHOR BOLT TENSIONING AND TESTING AS FOUND IN SECTION 4.E.2 VERIFYING THAT THE REQUIRED TENSION VALUE HAS BEEN ACHIEVED.

STATE OF OHIO
MATTHEW B. JOHNSON
E-74181
09/15/17
REGISTERED PROFESSIONAL ENGINEER

ISSUE FOR CONSTRUCTION
CONFIDENTIAL

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
A	KLT	CPB	CMM3	7/28/17	ISSUED FOR PROCUREMENT
0	RRR2	CMM3	MBJ	9/15/17	ISSUE FOR CONSTRUCTION

RELEASED TO/FOR	A	B	C	0	1	2	3
CLIENT	07/28/17						
BID							
CONSTRUCTION				09/15/17			
	DATE RELEASED						

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Scale	NONE
Date	3/17/2017
Drawn	KLT
Checked	CMM3
Designed	CPB
Approved	CMM3

WHITE CONSTRUCTION
CLINTON, INDIANA

NORTHWEST OHIO WIND PROJECT
PAULDING COUNTY, OHIO

SPREAD FOOTING FOUNDATION
TECHNICAL SPECIFICATIONS AND SUBMITTALS

BARR PROJECT No. 35631001.02
CLIENT PROJECT No. —
DWG. No. S-02
REV. No. 0

Foundation Design Computations
GE 2.5-116   90 Meter Hub Height
Northwest Ohio Wind Project
Starwood Energy Group Global
Paulding County, Ohio

Prepared for
White Construction Inc.
Clinton, Indiana

September 2017

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Contents

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 Ohio.

STATE OF OHIO
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E-74181
09/15/17
REGISTERED
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Matt B. Johnson
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September 15, 2017
Date

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

- Lukas Hartman, Structural Intern, Barr Engineering Company
- Alex Chizmadia, Structural Engineer, Barr Engineering Company
- Christopher Marr, Project Manager, Barr Engineering Company

I. Design References

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II. Basic Design Data

A. Material Properties

1. Concrete Strength

Minimum strength at 28 days, f'_c = 5,000 psi

2. Steel Reinforcing Yield Strength

Grade 75: f_y = 75,000 psi

Grade 60: f_y = 60,000 psi

3. Grout Strength

Minimum strength at 3 days, f'_c = 5,000 psi

Minimum strength at 28 days, f'_c = 9,500 psi

4. Embedment Plate Yield Strength

f_y = 36,000 psi

5. Anchor Bolt

Tensile Strength, f_u = 100,000 psi

Yield Strength, f_y = 75,000 psi

B. Foundation Loads Imposed by Tower and Turbine

The foundation has been designed according to the loads in Reference 3.

1. Reference 3 Load Accuracy

Barr's analysis and calculations rely on the foundation loading in Reference 3 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 and the actual loads are difficult or impossible to measure in the field, and can be measured, if at all, only after the construction is complete and under extreme circumstances. Thus, there is risk that the load data is incorrect or incomplete and Barr cannot provide assurance of foundation performance under other loads.

2. Design Fatigue Cycles

Twenty years based on fatigue loading provided in Reference 3.

C. Design Criteria

1. Minimum Factor of Safety Against Overturning

In accordance with generally accepted practices and standards of engineering and Reference 6, the summation of the moments of forces resisting overturning must be 1.5 times greater than the summation of the moments of forces causing overturning. This summation is taken at the base of the foundation.

2. Minimum Factor of Safety Against Sliding

In accordance with generally accepted practices and standards of engineering and Reference 6, the summation of forces resisting sliding must be 1.5 times greater than the summation of forces causing sliding. The summation is taken at the base of the foundation.

3. ACI 318 Load Factors and Strength Reduction Factors

In accordance with Chapter 9 of the ACI code (Reference 1a), the required strength (U) is computed by multiplying the service loads by load factors, which depend on the load type (wind, earthquake, live load, dead load) and the load combination being used. The wind loads are factored in accordance with Reference 10. The design strength of the element must be equal to or greater than the required strength. The design strength of the element is the nominal strength of the element times a strength reduction factor that depends on the type of force (shear, moment, bearing, compression) or combination of forces that are imparted on the element.

4. Minimum Factor of Safety Against Bearing Capacity Failure

In accordance with generally accepted practices and standards of engineering and Reference 6, for normal loading the typical factor of safety against a bearing capacity failure is 3.0 and a 133% increase in allowable bearing capacity is allowed for normal extreme wind loading. Hence, the factor of safety against a bearing capacity failure due to normal extreme wind loading is 3.0/1.33 or 2.25.

5. Minimum Foundation Stiffness

Foundation stiffness shall be in accordance with Reference 3 and Reference 9.

D. Soil Properties

1. Strength Data

Summary of Geotechnical Recommendations found in Reference 2.

2. Stiffness Properties

Summary of Geotechnical Recommendations found in Reference 2.

3. Design Groundwater Level

Summary of Geotechnical Recommendations found in Reference 2.

III. Foundation, Tower and Design Information

A. Unit Definitions and Foundation Dimensions

$k \equiv 1000 \cdot lbf$ $\quad$ $meter \equiv m$

Foundation width: $D := 61.5 \cdot ft$

Pedestal diameter: $C := 18 \cdot ft$

Average extension of pedestal above ground surface: $h_{pe} := 6in + \left(\frac{0.25in}{ft}\right) \cdot \left(\frac{D - C}{4}\right)$

$h_{pe} = 8.72 \cdot in$

Height of base: $h_b := 12 \cdot in$

Height of center: (above base) $h_c := 54 \cdot in$

Height of soil: (from foundation bottom) $h_s := 10.5ft - h_{pe}$

$h_s = 117.28 \cdot in$

Height of pedestal: $h_p := h_s - h_b - h_c + h_{pe}$

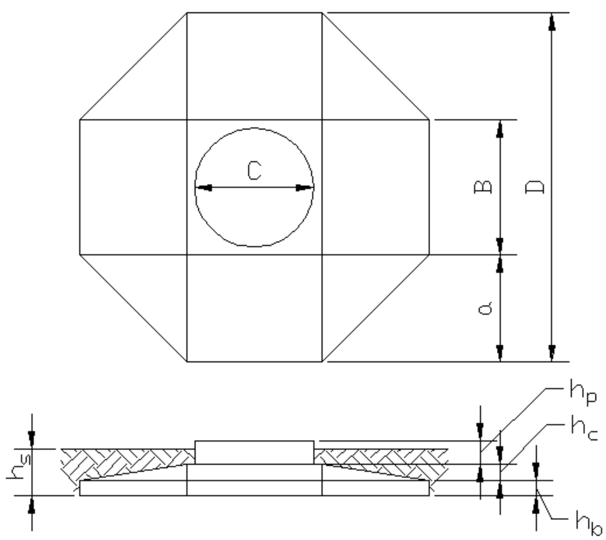
$h_p = 60.00 \cdot in$

Height of embedment ring above bottom of footing: $h_e := 27 \cdot in$

Minimum depth of groundwater below grade: $d_{GWT} := 2ft$

(Reference 2)

Minimum depth of groundwater below grade (fatigue): $d_{GWTF} := 2 \cdot ft$



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

$B = 25.47\,ft$

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

$a = 18.01\,ft$

Edge slope: $\xi_1 := \operatorname{atan}\left(\frac{h_c}{a}\right)$

$\xi_1 = 14.03 \cdot deg$

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

$\xi_2 = 19.46 \cdot deg$

Bottom cover: $cc_{bot} := 3 \cdot in$

Top cover: $cc_{top} := 2 \cdot in$

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

GroundwaterLocation = "Within the Pedestal"

B. Tower Bottom Flange Dimensions

Flange outside diameter: $OD := 4556 \cdot mm$ $OD = 14.95\,ft$

Flange inside diameter: $ID := 4000 \cdot mm$ $ID = 13.12\,ft$

Total number of bolts (2 circles): $N := 2 \cdot 70$

Inner bolt circle diameter: $D_i := 4130 \cdot mm$ $D_i = 13.55\,ft$

Outer bolt circle diameter: $D_o := 4426 \cdot mm$ $D_o = 14.52\,ft$

Average tower diameter: $d_{tower} := \frac{(OD + ID)}{2} = 14.04\,ft$ $d_{tower} = 14.04\,ft$

Thickness of tower flange: $t_{flange} := 75 \cdot mm$ $t_{flange} = 2.95 \cdot in$

Average bolt circle diameter: $D_{avg} := \frac{D_i + D_o}{2}$ $D_{avg} = 168.43 \cdot in$

Width of flange: $w_{flange} := \frac{OD - ID}{2}$ $w_{flange} = 10.94 \cdot in$

C. Stability Safety Factors

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

Minimum factor of safety: $FS_{min2} := 1.0$ (Reference 1)

D. Stiffness Requirements

Required dynamic rotational stiffness: $K_{\psi req} := \frac{5 \cdot 10^7 \cdot kN \cdot m}{rad}$ (Reference 3)

Required dynamic translational stiffness: $K_{xreq} := \frac{1 \times 10^6 \cdot kN}{m}$ (Reference 3)

E. ACI Reinforcing Information

(Reference 1a)

Bar nominal size, diameter (in), area (in^2), and weight (lbf/ft):

$$ACI\_bar\_table := \begin{pmatrix} \text{"not used"} & 0.0001 & 0 & 0 \\ 3 & 0.375 & 0.11 & 0.376 \\ 4 & 0.500 & 0.20 & 0.668 \\ 5 & 0.625 & 0.31 & 1.043 \\ 6 & 0.750 & 0.44 & 1.502 \\ 7 & 0.875 & 0.60 & 2.044 \\ 8 & 1.000 & 0.79 & 2.670 \\ 9 & 1.128 & 1.00 & 3.400 \\ 10 & 1.270 & 1.27 & 4.303 \\ 11 & 1.410 & 1.56 & 5.313 \\ 14 & 1.693 & 2.25 & 7.650 \\ 18 & 2.257 & 4.00 & 13.600 \end{pmatrix}$$

F. Material Properties

Friction factor: $\mu_f := 0.4$ (Reference 2)

Concrete strength: $f_c := 5000 \cdot psi$

Steel yield strength: $f_y := 75000psi$

Steel verts yield strength: $f_{yv} := 60000 \cdot psi$

Steel modulus of elasticity: $E_s := 29000ksi$

Density of concrete: $\gamma_c := 150pcf$

Density of water: $\gamma_w := 62.4pcf$

Design density of soil above GWT for bottom steel and stability: $\gamma_{sdbot} := 110pcf$ (Reference 2)

Design density of soil below GWT for bottom steel and stability: $\gamma_{ssbot} := 125pcf$

Design density of soil above GWT for top steel: $\gamma_{sdtop} := 125pcf$

Design density of soil below GWT for top steel: $\gamma_{sstop} := 125pcf$

Soil wedge angle from vertical: $\theta := atan\left(\frac{1}{2}\right)$ $\theta = 26.6 \cdot deg$

Soil wedge angle from vertical (fatigue): $\theta_{fat} := atan\left(\frac{0}{2}\right)$ $\theta_{fat} = 0.00 \cdot deg$

Concrete modulus of elasticity: $E_c := 57000psi \cdot \sqrt{\frac{f_c}{psi}}$ $E_c = 4031 \cdot ksi$

Modulus reduction factor: $\psi := 0.8$ (Reference 7)

Modular ratio: $n_{mod} := \frac{E_s}{\psi \cdot E_c}$ $n_{mod} = 9.0$

G. Extreme Loading Conditions

(Reference 3)

Misalignment Loading (Section 3)

Base moment: $M_{align} := 1335kN \cdot m$

Misalignment angle (relative to wind):

$\Delta := 0deg$

$M_{align} = 985 \cdot ft \cdot k$

Design Load Case 1.3: Normal Extreme

Normal extreme load factor:

$\alpha_e := 1.4$

Base moment:

$M_e := \frac{60753.7}{1.35} kN \cdot m = 45003 \cdot kN \cdot m$

$M_e = 33192 \cdot ft \cdot k$

Base shear:

$H_e := \frac{691.7}{1.35} kN = 512 \cdot kN$

$H_e = 115 \cdot k$

Tower & turbine dead weight:

$W_{te} := \frac{3643.4}{1.35} kN = 2699 \cdot kN$

$W_{te} = 607 \cdot k$

Design Load Case 6.2: Abnormal Extreme

Abnormal extreme load factor:

$\alpha_a := 1.14$

Base moment:

$M_a := \frac{61262.32}{1.10} kN \cdot m$

$M_a = 41077 \cdot k \cdot ft$

Base shear:

$H_a := \frac{760.7}{1.10} kN$

$H_a = 155 \cdot k$

Tower & turbine dead weight:

$W_{ta} := \frac{2887.1}{1.10} kN$

$W_{ta} = 590 \cdot k$

H. Normal Loading Conditions - DLC 1.0

Base moment:

$M_N := 35477.7 \cdot kN \cdot m$

$M_N = 26167 \cdot ft \cdot k$

Base shear:

$H_N := 410.6 \cdot kN$

$H_N = 92 \cdot k$

Tower & turbine dead weight:

$W_N := 2708.7 \cdot kN$

$W_N = 609 \cdot k$

I. Normal Loading Conditions - DLC 1.3

Base moment:

$M_{1.1} := 39454.6 \cdot kN \cdot m$

$M_{1.1} = 29100 \cdot ft \cdot k$

Base shear:

$H_{1.1} := 462.4 \cdot kN$

$H_{1.1} = 104 \cdot k$

Tower & turbine dead weight:

$W_{1.1} := 2722.6 \cdot kN$

$W_{1.1} = 612 \cdot k$

J. Earthquake Loading Conditions

(Reference 1)

Seismic Design Criteria:

$Site_{Class} := \text{"C"}$ (1600 ft/s shear wave velocity - Reference 2)

- Building Occupancy Category: II (Non-Essential Power Facility, Non Hazardous)
- Seismic Design Category: B (Determination of Seismic Design Category)

0.2 Second spectral response: $S_S := 0.15$ (USGS: Paulding County Ohio)

1.0 Second spectral response: $S_1 := 0.07$ (USGS: Paulding County Ohio)

Table 11.4-1 Site Coefficient, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at Short Period				
	Ss <= 0.25	Ss = 0.50	Ss = 0.75	Ss = 1.00	Ss >= 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				

Table 11.4-2 Site Coefficient, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration parameter at 1-s Period				
	Ss <= 0.10	Ss = 0.20	Ss = 0.30	Ss = 0.40	Ss >= 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				

$F_a = 1.20$ $F_v = 1.70$

Importance factor: $I := 1.00$

Adjusted 0.2s response: $S_{MS} := F_a \cdot S_S$ $S_{MS} = 0.18$

Adjusted 1.0s response: $S_{M1} := F_v \cdot S_1$ $S_{M1} = 0.12$

Design response: $S_{DS} := \frac{2}{3} \cdot S_{MS}$ $S_{DS} = 0.12$

$S_{D1} := \frac{2}{3} \cdot S_{M1}$ $S_{D1} = 0.08$

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

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: $h_n := 90 \cdot m$

Approximate fundamental period of structure: $T_{a1} := C_t \cdot \left(\frac{h_n}{ft}\right)^{x_{EQ}} \cdot s$ $\quad T_{a1} = 1.42\,s$

Value of Cu 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 Cu
>= 0.4	1.4
0.3	1.4
0.2	1.5
0.15	1.6
<= 0.1	1.7

Maximum approximate period: $T_{a2} := C_u \cdot T_{a1}$ $\quad T_{a2} = 2.42\,s$

Design period: $T := \min(T_{a1}, T_{a2})$ $\quad T = 1.42\,s$

Seismic response coefficients: $C_{s1} := \frac{S_{DS}}{\frac{R}{I}}$ $\quad C_{s1} = 0.08$

$$C_{s2} := \text{if}\left[\frac{T}{sec} \le T_L, \frac{S_{D1}}{\left(\frac{T}{sec}\right)\cdot\left(\frac{R}{I}\right)}, \frac{S_{D1}\cdot T_L}{\left(\frac{T}{sec}\right)^2\cdot\left(\frac{R}{I}\right)}\right] = 0.037$$

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

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

$C_s := \max(\min(C_{s1}, C_{s2}), C_{s3}, C_{s4})$ $\quad C_s = 0.037$

Minimum base shear: $V := C_s \cdot W_{1.1}$ $\quad V = 23 \cdot kip$

Structure Weights and Centers of Gravity:

(Reference 14)

Total weight of structure: $W_{1.1} = 612 \cdot kip$

	Weight of Component	Approximate Center of Gravity
Tower bottom section::	$W_1 := 43954kgf$	$h_1 := \frac{32.463}{2} m = 16231.50 \cdot mm$
Tower lower mid section:	$W_2 := 55091kgf$	$h_2 := 32.463m + \frac{31.600}{2} m = 48263.00 \cdot (mm)$
Tower upper mid section:	$W_3 := 62954kgf$	$h_3 := 32.463m + 31.600m + \frac{23.342m}{2} = 75734.00 \cdot (mm)$
Tower top section:	$W_4 := 0kgf$	$h_4 := 0$
Wind turbine nacelle and rotor:		$h_5 := 90 \cdot m$

Total tower weight: $W_{twr} := W_1 + W_2 + W_3 + W_4$ $\quad W_{twr} = 357 \cdot kip$

Wind turbine nacelle and rotor: $W_5 := W_{1.1} - W_{twr}$ $\quad W_5 = 255 \cdot kip$

Summary:

$$W = \begin{pmatrix} 0 \\ 97 \\ 121 \\ 139 \\ 0 \\ 255 \end{pmatrix} \cdot kip \qquad h = \begin{pmatrix} 0.0 \\ 16.2 \\ 48.3 \\ 75.7 \\ 0.0 \\ 90.0 \end{pmatrix} \cdot m$$

Determine Design Base Shear:

Exponent related to period of structure:

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

$$k_0 = 1.46$$

Number of tower components: $ntc := 1..5$

Vertical Distribution Factor:

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

Lateral Seismic Force at Each Level:

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

$C_{v_{ntc}} =$

0.019
0.116
0.257
0.000
0.608

$F_{ntc} =$ $\cdot kip$

0.4
2.6
5.8
0.0
13.8

Total lateral seismic force: $H_{EQ} := \sum_{i=1}^{5}(F_i)$ $H_{EQ} = 23 \cdot kip$

Earthquake overturning moment: $M_{EQ} := \sum_{i=1}^{5}(F_i \cdot h_i)$ $M_{EQ} = 5970 \cdot ft \cdot kip$

Base moment: $M_{OE} := \sqrt{M_{1.1}^2 + M_{EQ}^2}$ $M_{OE} = 29706 \cdot ft \cdot k$

Base shear: $H_{OE} := \sqrt{H_{1.1}^2 + H_{EQ}^2}$ $H_{OE} = 106 \cdot k$

Tower & turbine dead weight: $W_{OE} := W_{1.1}$ $W_{OE} = 612 \cdot k$

K. Fatigue Loading Conditions

(Reference 3)

Mean shear: $H_{mean} := 344.1kN$ $H_{mean} = 77 \cdot kip$

Mean overturning moment: $M_{mean} := 30130kN \cdot m$ $M_{mean} = 22223 \cdot kip \cdot ft$

Turbine & tower mean weight: $W_{mean} := 2689(kN)$ $W_{mean} = 605 \cdot kip$

Fatigue Loading (Markov Matrix):

Number of Cycles	Overturning Moment (kN-m)	
	Min	Max
120	0	0
100	0	0
100	0	0
4936	0	0
1000	0	0
40	0	0
40	0	0
1414	0	0
4936	0	0
15500	0	0
2828	0	0
2828	0	0
1414	0	0
1530	0	0
3919000	0	0
136400	0	0
6366	0	0
4780	0	0
11390	0	0
2536	0	0
193	0	0
8835	0	0
1630	0	0
16070	0	0
6845	0	0
9620	0	100
6076	100	300

Data Rows
10812

Bin counters:

$qt := NRows - 1 = 10811$

$qr := 0, 1 .. qt$

$Years_{Design} := 20$ $Years_{Matrix} := 20$

$N_{fat_{qr}} := N_{fat_{qr}} \cdot Years_{Design} \div Years_{Matrix}$

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

Minimum Moment (for Miner's Rule):

$M_{fatminnorth_{qr}} := MinBin_{qr} \cdot kN \cdot m$

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

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

Maximum Moment (for Miner's Rule):

$M_{fatmaxnorth_{qr}} := MaxBin_{qr} \cdot kN \cdot m$

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

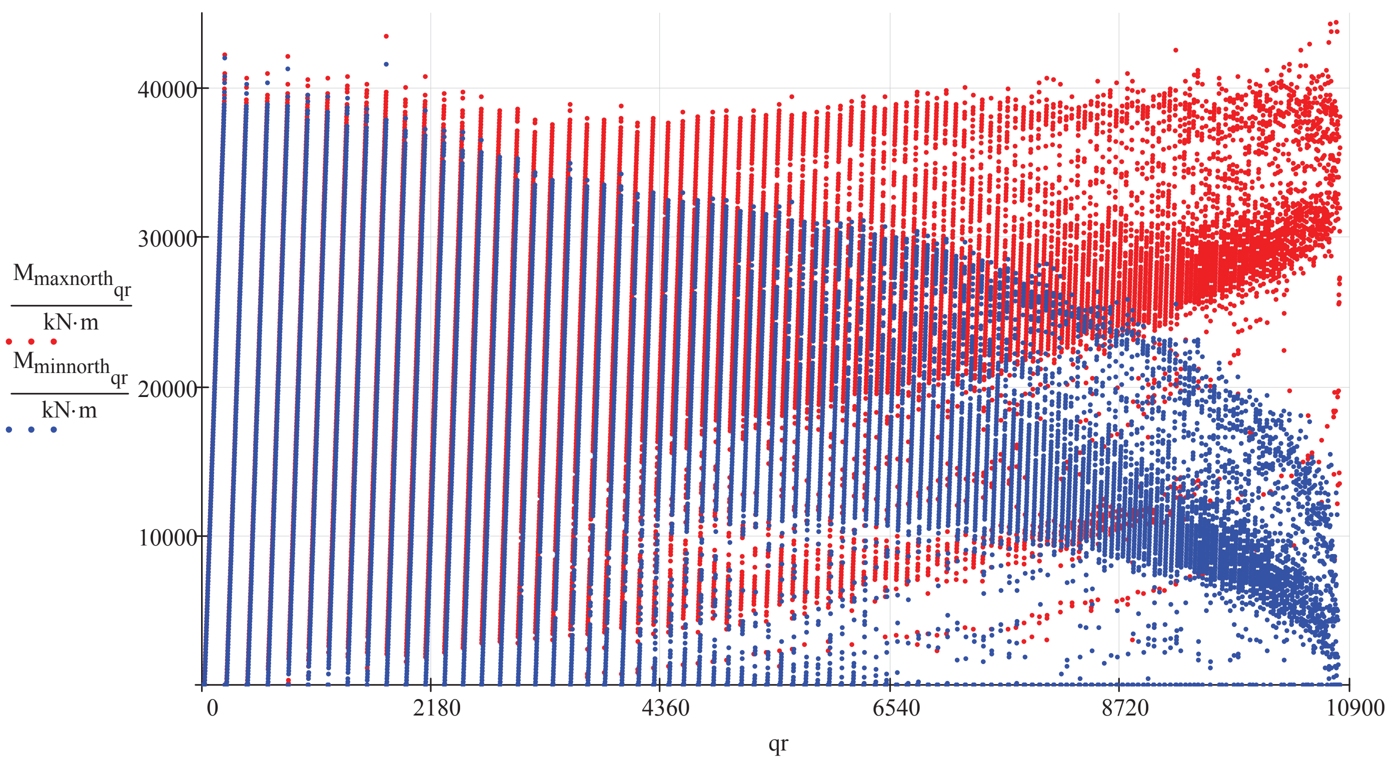
$$M_{maxnorth_{qr}} := M_{fatmaxnorth_{qr}} + H_{maxnorth_{qr}} \cdot \left(h_p + h_c + h_b\right)$$

Maximum Moments:

$M_{maxfatigue} := \max\left(M_{maxnorth}\right)$ $M_{maxfatigue} = 44364 \cdot kN \cdot m$

$M_{minfatigue} := \max\left(M_{minnorth}\right)$ $M_{minfatigue} = 41980 \cdot kN \cdot m$

Fatigue moments are plotted below:

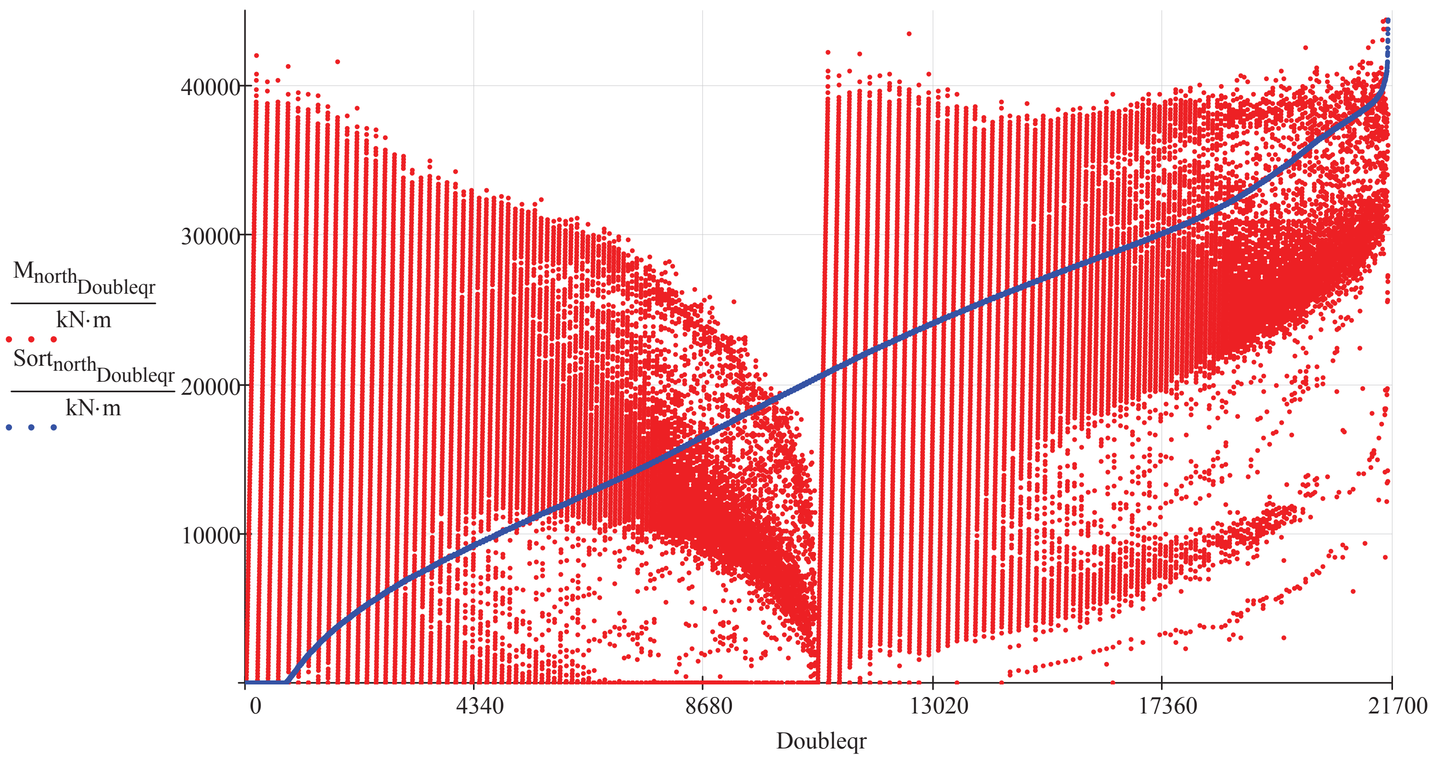


L. Stack Min and Max Fatigue Matrices and Sort

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

$$M_{north_{Doubleqr}} := \begin{cases} M_{minnorth_{Doubleqr}} & \text{if } Doubleqr \le qt \\ M_{maxnorth_{Doubleqr-qt-1}} & \text{if } Doubleqr > qt \end{cases}$$

$$Sort_{north} := sort\left(M_{north}\right)$$



M. Develop Unique Fatigue Matrix and Parse Zero Values

Develop matrix containing only the unique fatigue values:

$$Unique_{north_{Doubleqr}} := \begin{cases} Sort_{north_0} & \text{if } Doubleqr = 0 \\ Sort_{north_{Doubleqr}} & \text{if } Doubleqr \neq 0 \wedge Sort_{north_{Doubleqr}} \neq Sort_{north_{Doubleqr-1}} \\ 0 & \text{if } Doubleqr \neq 0 \wedge Sort_{north_{Doubleqr}} = Sort_{north_{Doubleqr-1}} \end{cases}$$

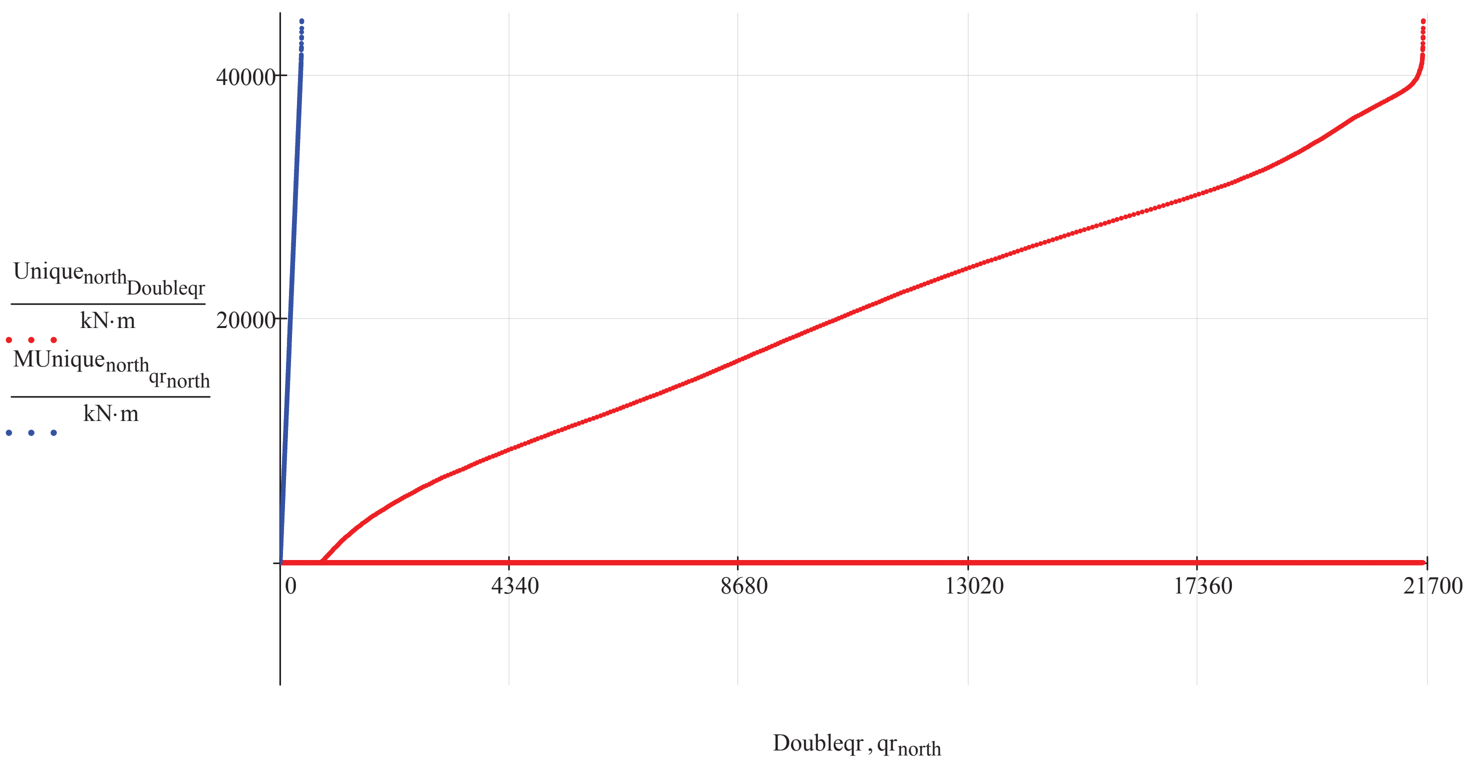
Remove all "zero" entires leaving only "non-zero" entries in the MUnique matrix:

$$MUnique_{north} := \begin{cases} trim\left(Unique_{north}, match\left(0, Unique_{north}\right)\right) & \text{if } \min\left(Unique_{north}\right) = 0 \\ Unique_{north} & \text{if } \min\left(Unique_{north}\right) > 0 \end{cases}$$

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

$$qt_{north} := length\left(MUnique_{north}\right) - 1 = 410$$

$$qr_{north} := 0, 1 .. qt_{north}$$



IV. Stability Analysis and Bearing Length Evaluation

A. Foundation Volume and Weight Calculations

Foundation plan area: $A := D^2 - 2\cdot\left(\frac{D-B}{2}\right)^2$ $A = 3133\cdot ft^2$

Volume of pedestal: $v_p := \frac{\pi\cdot C^2}{4}\cdot h_p$ $v_p = 47\cdot yd^3$

Weight of pedestal: $W_p := v_p\cdot\gamma_c$ $W_p = 191\cdot k$

Volume of footing: $v_f := A\cdot h_b + B^2\cdot h_c + 4\cdot\left(\frac{1}{2}\cdot\frac{1}{3}\cdot h_c\cdot a^2 + \frac{1}{2}\cdot B\cdot h_c\cdot a\right)$ $v_f = 413\cdot yd^3$

Weight of footing: $W_f := v_f\cdot\gamma_c$ $W_f = 1674\cdot k$

Total volume of concrete: $v_c := v_f + v_p$ $v_c = 460\cdot yd^3$

Total volume of soil:

$$v_s := A\cdot h_s - v_f - \frac{\pi\cdot C^2}{4}\cdot\left(h_s - h_b - h_c\right) \ldots + \frac{8\cdot B\cdot\tan(\theta)}{2}\cdot\left(h_s - h_b\right)^2$$

$v_s = 826\cdot yd^3$

Total volume of soil (fatigue)

$$v_{sfat} := A\cdot h_s - v_f - \frac{\pi\cdot C^2}{4}\cdot\left(h_s - h_b - h_c\right) \ldots + \frac{8\cdot B\cdot\tan\left(\theta_{fat}\right)}{2}\cdot\left(h_s - h_b\right)^2$$

$v_{sfat} = 681\cdot yd^3$

Equivalent buoyant force:

$$F_b := \gamma_w\cdot \begin{cases} \text{if } d_{GWT} < h_s & \begin{cases} A\cdot\left(h_s - d_{GWT}\right) & \text{if } d_{GWT} \ge h_s - h_b \\ A\cdot\left(h_s - d_{GWT}\right) + \frac{8\cdot B\cdot\tan(\theta)}{2}\cdot\left(h_s - h_b - d_{GWT}\right)^2 & \text{otherwise} \end{cases} \\ 0 & \text{otherwise} \end{cases}$$

$F_b = 1666\cdot kip$

Equivalent buoyant force (fatigue):

$$F_{bfat} := \gamma_w\cdot \begin{cases} \text{if } d_{GWTF} < h_s & \begin{cases} A\cdot\left(h_s - d_{GWTF}\right) & \text{if } d_{GWTF} \ge h_s - h_b \\ A\cdot\left(h_s - d_{GWTF}\right) + \frac{8\cdot B\cdot\tan\left(\theta_{fat}\right)}{2}\cdot\left(h_s - h_b - d_{GWTF}\right)^2 & \text{otherwise} \end{cases} \\ 0 & \text{otherwise} \end{cases}$$

$F_{bfat} = 1520\cdot kip$

Total weight of soil:

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

$$W_s = 2677 \cdot k$$

Total weight of soil (fatigue):

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

$W_{sfat} = 2211 \cdot k$

Total dead weight seismic load: $W_{EQ} := W_p + W_f + W_{OE} + W_s - F_b$ $W_{EQ} = 3488 \cdot k$

Total dead weight fatigue load: $W_{fat} := W_p + W_f + W_{mean} + W_{sfat} - F_{bfat}$ $W_{fat} = 3160 \cdot k$

B. Stability Calculations - Extreme Loading

Determine controlling extreme load case: $\boxed{\text{if}\left(M_e > M_a, \text{"Normal Controls"}, \text{"Abnormal Controls"}\right) = \text{"Abnormal Controls"}}$

Extreme Tower Base Moment: $M := \sqrt{\left(\text{if}\left(M_e > M_a, M_e, M_a\right) + M_{align} \cdot \cos(\Delta)\right)^2 + \left(M_{align} \cdot \sin(\Delta)\right)^2} = 42062 \cdot k \cdot ft$

Extreme Tower Base Shear: $H := \text{if}\left(M_e > M_a, H_e, H_a\right)$ $\quad H = 155 \cdot k$

Extreme Tower Weight: $W_t := \text{if}\left(M_e > M_a, W_{te}, W_{ta}\right)$ $\quad W_t = 590 \cdot k$

Total dead weight wind load: $W_W := W_p + W_f + W_t + W_s - F_b$ $\quad W_W = 3466 \cdot k$

Overturning wind moment: $M_{oW} := M + \left(h_b + h_c + h_p\right) \cdot H$ $\quad M_{oW} = 43694 \cdot k \cdot ft$

Wind load friction resistance at base: $H_{frW} := \mu_f \cdot \left(W_W\right)$ $\quad H_{frW} = 1386 \cdot k$

Factor of safety against sliding: $FS_{sW} := \dfrac{H_{frW}}{H}$ $\quad FS_{sW} = 8.92$

Seismic load friction resistance at base: $H_{frEQ} := \mu_f \cdot \left(W_{EQ}\right) \cdot \left(1 - E_v\right)$ $\quad H_{frEQ} = 1395 \cdot k$

Factor of safety against sliding: $FS_{sEQ} := \dfrac{H_{frEQ}}{H_{OE}}$ $\quad FS_{sEQ} = 13.11$

Determine controlling load case: $FS_s := \min\left(FS_{sW}, FS_{sEQ}\right)$ $\quad FS_s = 8.92$

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

Resisting moment: $M_{rW} := W_W \cdot \min\left(\dfrac{D}{2}, \dfrac{D - a}{\sqrt{2}}\right)$ $\quad M_{rW} = 106584 \cdot ft \cdot k$

Factor of safety against overturning: $FS_{oW} := \dfrac{M_{rW}}{M_{oW}}$ $\quad FS_{oW} = 2.44$

Overturning seismic moment: $M_{oEQ} := \sqrt{\left(M_{OE} + M_{align} \cdot \cos(\Delta)\right)^2 + \left(M_{align} \cdot \sin(\Delta)\right)^2} + \left(h_b + h_c + h_p\right) \cdot H_{OE} = 31808 \cdot k \cdot ft$

Resisting moment: $M_{rEQ} := \left(1 - E_v\right) \cdot W_{EQ} \cdot \min\left(\dfrac{D}{2}, \dfrac{D - a}{\sqrt{2}}\right)$ $\quad M_{rEQ} = 107261 \cdot ft \cdot k$

Factor of safety against overturning: $FS_{oEQ} := \dfrac{M_{rEQ}}{M_{oEQ}}$ $\quad FS_{oEQ} = 3.37$

Minimum factor of safety: $FS_{min} = 1.50$

Determine controlling load case: $FS_o := \min\left(FS_{oW}, FS_{oEQ}\right)$ $\quad FS_o = 2.44$

$\boxed{Output_{Overturning} := FS_o}$

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

Resisting moment (reduced): $M_{rW\_red} := 0.6W_W \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$ $M_{rW\_red} = 63951 \cdot ft \cdot k$

Factor of safety against overturning (alternate): $FS_{oW\_alt} := \frac{M_{rW\_red}}{M_{oW}}$ $FS_{oW\_alt} = 1.46$

Overturning seismic moment (reduced): $M_{oEQ\_alt} := 0.7 \cdot M_{oEQ}$ $M_{oEQ\_alt} = 22266 \cdot k \cdot ft$

Resisting moment (reduced): $M_{rEQ\_red} := 0.6\left(1 - E_v\right) \cdot W_{EQ} \cdot \min\left(\frac{D}{2}, \frac{D-a}{\sqrt{2}}\right)$ $M_{rEQ\_red} = 64357 \cdot ft \cdot k$

Factor of safety against overturning (alternate): $FS_{oEQ\_alt} := \frac{M_{rEQ\_red}}{M_{oEQ\_alt}}$ $FS_{oEQ\_alt} = 2.89$

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

Determine controlling load case: $FS_{o2} := \min\left(FS_{oW\_alt}, FS_{oEQ\_alt}\right)$ $FS_{o2} = 1.46$ $\boxed{Output_{Overturning2} := FS_{o2}}$

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

C. Soil Pressure Calculations - Extreme Loading

Side length of square inscribed inside of foundation octagon: $S_f := \sqrt{2a^2 + 2 \cdot a \cdot B + B^2}$ $S_f = 47.1\,ft$

"Major" width of octagon along widest section: $H_f := \sqrt{B^2 + D^2}$ $H_f = 66.6\,ft$

Distance from widest section of octagon to edge of square: $W_e := \frac{H_f - S_f}{2}$ $W_e = 9.75\,ft$

Moment of inertia for octagon about any axis through centroid: $I_{fdn} := \frac{8 \cdot B^4}{192} \cdot \cot\left(\frac{2 \cdot \pi}{16}\right)\left(3\cot\left(\frac{2 \cdot \pi}{16}\right)^2 + 1\right)$ $I_{fdn} = 783048 \cdot ft^4$

Section modulus of foundation for normal orientation: $S_{normal} := \frac{2I_{fdn}}{D}$ $S_{normal} = 25465 \cdot ft^3$

Section modulus of foundation for orientation rotated by 22.5 degrees: $S_{rotated} := \frac{2 \cdot I_{fdn}}{H_f}$ $S_{rotated} = 23527 \cdot ft^3$

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

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}

$F := W_W$ $\quad F = 3466 \cdot k$ $\quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW}$ $\quad M_{toe} = 71672 \cdot ft \cdot k$

Guess: $\quad L := \frac{2W_e + H_f}{4}$ $\quad f_{max} := 3172 \cdot psf$

Given

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

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

$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \text{Find}(L, f_{max})$ $\quad L_4 = 48.5\,ft$ $\quad f_4 = 3172 \cdot psf$

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

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

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}

$F := W_W \qquad F = 3466 \cdot k \qquad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \qquad M_{toe} = 71672 \cdot ft \cdot k$

Guess: $L := \frac{3H_f - 2W_e}{4} \qquad f_{max} := 3138 \cdot psf$

Given

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

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

$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := \mathrm{Find}(L, f_{max}) \qquad L_5 = 49.6\,ft \qquad f_5 = 3138 \cdot psf$

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: $L_{b5} := \mathrm{if}\left(L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, L_5, 0 \cdot ft\right) \qquad L_{b5} = 49.6 \cdot ft$

Maximum soil bearing pressure: $f_{max5} := \mathrm{if}\left(L_5 < H_f - W_e \wedge L_5 > \frac{H_f}{2}, f_5, 0 \cdot psf\right) \qquad f_{max5} = 3138 \cdot psf$

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}

$F := W_W \qquad F = 3466 \cdot k \qquad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oW} \qquad M_{toe} = 71672 \cdot ft \cdot k$

Guess: $\qquad L := \frac{2H_f - W_e}{2} \qquad f_{max} := 3166 \cdot psf$

Given

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

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

$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \qquad L_6 = 49.0\,ft \qquad f_6 = 3166 \cdot psf$

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

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

4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing (L_b) equal to the 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).

$F := W_W$ $\quad F = 3466 \cdot k$ $\quad M_{toe} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{oW}$ $\quad M_{toe} = 71672 \cdot ft \cdot k$

Guess: $\quad f_{max} := 2963 \cdot psf$ $\quad df := 3714 \cdot psf$

Given

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

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

$\begin{pmatrix} df \\ f_{max} \end{pmatrix} := \text{Find}(df, f_{max})$ $\quad f_{max} = 2963 \cdot psf$ $\quad df = 3714 \cdot psf$

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

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

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

D. Bearing Length Check - Extreme Loading

Select Bearing Length and Pressure Distribution

Bearing length: $L_{bW} := \text{if}\left(f_{max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6}\right)$ $L_{bW} = 49.6 \cdot ft$

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

$f_{maxW} = 3138 \cdot psf$

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

Determine controlling load case: $L_b := L_{bW}$ $L_b = 49.6\,ft$

Maximum soil bearing pressure: $f_{max} := f_{maxW}$ $f_{max} = 3138 \cdot psf$

Minimum soil bearing pressure: $f_{min} := f_{minW}$ $f_{min} = 0 \cdot psf$

Percent of base by length in compression under extreme loading: $\frac{L_b}{H_f} = 0.74$

$\boxed{\text{if}\left(\frac{L_b}{H_f} \geq 0.5, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}}$ (Reference 10)

E. Foundation Volume and Weight Calculations - Normal Loading

Total dead weight: $W_{totN} := W_p + W_f + W_s + W_N - F_b$ $\quad W_{totN} = 3485 \cdot k$

Overturning moment: $M_{oN} := \sqrt{\left(M_N + M_{align} \cdot \cos(\Delta)\right)^2 + \left(M_{align} \cdot \sin(\Delta)\right)^2} + \left(h_b + h_c + h_p\right) \cdot H_N = 28121 \cdot k \cdot ft$

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

$F := W_{totN}$ $\quad F = 3485 \cdot k$ $\quad M_{toe} := F \cdot \dfrac{\sqrt{B^2 + D^2}}{2} - M_{oN}$ $\quad M_{toe} = 87874 \cdot ft \cdot k$

Guess: $L := \dfrac{2W_e + H_f}{4}$ $\quad f_{max} := 2366 \cdot psf$

Given

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

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

$\begin{pmatrix} L_4 \\ f_4 \end{pmatrix} := \mathrm{Find}\left(L, f_{max}\right)$ $\quad L_4 = 59.6\,ft$ $\quad f_4 = 2366 \cdot psf$

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: $L_{b4} := \mathrm{if}\left(L_4 \geq W_e \wedge L_4 \leq \dfrac{H_f}{2}, L_4, 0 \cdot ft\right)$ $\quad L_{b4} = 0.0 \cdot ft$

Maximum soil bearing pressure: $f_{max4} := \mathrm{if}\left(L_4 \geq W_e \wedge L_4 \leq \dfrac{H_f}{2}, f_4, 0 \cdot psf\right)$ $\quad f_{max4} = 0 \cdot psf$

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

$F := W_{totN}$ $F = 3485 \cdot k$ $M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN}$ $M_{toe} = 87874 \cdot ft \cdot k$

Guess: $L := \frac{3H_f - 2W_e}{4}$ $f_{max} := 2315 \cdot psf$

Given

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

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

$\begin{pmatrix} L_5 \\ f_5 \end{pmatrix} := Find(L, f_{max})$ $L_5 = 63.9\,ft$ $f_5 = 2315 \cdot psf$

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

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

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}

$F := W_{totN}$ $\quad F = 3485 \cdot k \quad M_{toe} := F \cdot \frac{\sqrt{B^2 + D^2}}{2} - M_{oN} \quad M_{toe} = 87874 \cdot ft \cdot k$

Guess: $\quad L := \frac{2H_f - W_e}{2} \quad f_{max} := 2308 \cdot psf$

Given

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

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

$\begin{pmatrix} L_6 \\ f_6 \end{pmatrix} := \text{Minerr}(L, f_{max}) \quad L_6 = 64.2\,ft \quad f_6 = 2308 \cdot psf$

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: $\quad L_{b6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, L_6, 0 \cdot ft) \quad L_{b6} = 64.2 \cdot ft$

Maximum soil bearing pressure: $\quad f_{max6} := \text{if}(L_6 < H_f \wedge L_6 > H_f - W_e, f_6, 0 \cdot psf) \quad f_{max6} = 2308 \cdot psf$

4) Assumed Soil Pressure Case 4

Assume trapezoidal distribution with length of bearing (L_b) equal to the 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).

$F := W_{totN}$ $\qquad F = 3485 \cdot k$ $\qquad M_{toe} := \frac{F \cdot \sqrt{B^2 + D^2}}{2} - M_{oN}$ $\qquad M_{toe} = 87874 \cdot ft \cdot k$

Guess: $\qquad f_{max} := 2308 \cdot psf$ $\qquad df := 2391 \cdot psf$

Given

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

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

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

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

$\begin{pmatrix} df \\ f_{max} \end{pmatrix} := \mathrm{Find}(df, f_{max})$ $\qquad f_{max} = 2308 \cdot psf$ $\qquad df = 2391 \cdot psf$

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

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

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

G. Bearing Length Check - Normal Loading

Select Bearing Length and Pressure Distribution

Bearing length: $L_{bN} := \text{if}\left(f_{max7} > 0, H_f, L_{b4} + L_{b5} + L_{b6}\right)$ $\quad L_{bN} = 64.2 \cdot ft$

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

$f_{max} = 2308 \cdot psf$

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

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

$A_N = 3120 \cdot ft^2$

Percent of base by area in compression under normal loading: $\frac{A_N}{A} = 99.6 \cdot \%$

$\boxed{\text{if}\left(\frac{A_N}{A} \geq 0.994, \text{"OK"}, \text{"No Good"}\right) = \text{"OK"}}$ (Reference 10)

V. Bearing Capacity Evaluation

(Reference 8)

A. Design Soil Bearing Pressure - Normal Loading

Design overturning moment:	$M_{dN} := M_{oN}$	$M_{dN} = 28121 \cdot ft \cdot k$
Design vertical load:	$V_{dN} := W_{totN}$	$V_{dN} = 3485 \cdot k$
Design load eccentricity:	$e_{dN} := \dfrac{M_{dN}}{V_{dN}}$	$e_{dN} = 8.07\,ft$
Circular radius of octagon:	$R := \dfrac{D}{2}$	$R = 30.75\,ft$
Effective soil area in bearing:	$A_{effN} := 2 \cdot \left[\left(R^2\right) \cdot acos\left(\dfrac{e_{dN}}{R}\right) - e_{dN} \cdot \sqrt{R^2 - e_{dN}{}^2}\right]$	$A_{effN} = 1990\,ft^2$
Ellipse soil width in bearing:	$b_{eN} := 2 \cdot \left(R - e_{dN}\right)$	$b_{eN} = 45.4\,ft$
Ellipse soil length in bearing:	$l_{eN} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \dfrac{b_{eN}}{2 \cdot R}\right)^2}$	$l_{eN} = 59.3\,ft$
Effective soil length in bearing:	$l_{effN} := \sqrt{A_{effN} \cdot \dfrac{l_{eN}}{b_{eN}}}$	$l_{effN} = 51.0\,ft$
Effective soil width in bearing:	$b_{effN} := \dfrac{l_{effN}}{l_{eN}} \cdot b_{eN}$	$b_{effN} = 39.0\,ft$
Design bearing pressure:	$f_{dN} := \dfrac{V_{dN}}{A_{effN}}$	$f_{dN} = 1752 \cdot psf$

B. Bearing Capacity Check - Normal Loading

Allowable bearing pressure:	$f_{all\_N} := 3300psf$	(Reference 2)
Ratio of design bearing pressure to allowable bearing pressure:	$\boxed{\dfrac{f_{dN}}{f_{all\_N}} = 0.53}$	

C. Design Soil Bearing Pressure - Extreme Normal Loading

Design overturning moment: $M_{dW} := \sqrt{\left(M_e + M_{align} \cdot \cos(\Delta)\right)^2 + \left(M_{align} \cdot \sin(\Delta)\right)^2} + \left(h_b + h_c + h_p\right) \cdot H_e = 35386 \cdot k \cdot ft$

Design vertical load: $V_{dW} := W_p + W_f + W_{te} + W_s - F_b$ $\qquad V_{dW} = 3483 \cdot k$

Design load eccentricity: $e_{dW} := \dfrac{M_{dW}}{V_{dW}}$ $\qquad e_{dW} = 10.16\,ft$

Circular radius of octagon: $R = 30.75\,ft$

Effective soil area in bearing: $A_{effW} := 2 \cdot \left[\left(R^2\right) \cdot \operatorname{acos}\left(\dfrac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}{}^2}\right]$ $\qquad A_{effW} = 1744\,ft^2$

Ellipse soil width in bearing: $b_{eW} := 2 \cdot \left(R - e_{dW}\right)$ $\qquad b_{eW} = 41.2\,ft$

Ellipse soil length in bearing: $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \dfrac{b_{eW}}{2 \cdot R}\right)^2}$ $\qquad l_{eW} = 58.0\,ft$

Effective soil length in bearing: $l_{effW} := \sqrt{A_{effW} \cdot \dfrac{l_{eW}}{b_{eW}}}$ $\qquad l_{effW} = 49.6\,ft$

Effective soil width in bearing: $b_{effW} := \dfrac{l_{effW}}{l_{eW}} \cdot b_{eW}$ $\qquad b_{effW} = 35.2\,ft$

Design bearing pressure: $f_{dW} := \dfrac{V_{dW}}{A_{effW}}$ $\qquad f_{dW} = 1997 \cdot psf$

D. Bearing Capacity Check - Extreme Normal Loading

Allowable bearing pressure: $f_{allW} := 4200(psf)$ $\qquad$ (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure: $\boxed{\dfrac{f_{dW}}{f_{allW}} = 0.48}$

E. Design Soil Bearing Pressure - Extreme Abnormal Loading

Design overturning moment: $M_{dA} := \sqrt{\left(M_a + M_{align} \cdot \cos(\Delta)\right)^2 + \left(M_{align} \cdot \sin(\Delta)\right)^2} + \left(h_b + h_c + h_p\right) \cdot H_a = 43694 \cdot k \cdot ft$

Design vertical load: $V_{dA} := W_p + W_f + W_{ta} + W_s - F_b$ $\quad V_{dA} = 3466 \cdot k$

Design load eccentricity: $e_{dA} := \dfrac{M_{dA}}{V_{dA}}$ $\quad e_{dA} = 12.61\,ft$

Circular radius of octagon: $R = 30.75\,ft$

Effective soil area in bearing: $A_{effA} := 2 \cdot \left[\left(R^2\right) \cdot acos\left(\dfrac{e_{dA}}{R}\right) - e_{dA} \cdot \sqrt{R^2 - e_{dA}^2}\right]$ $\quad A_{effA} = 1465\,ft^2$

Ellipse soil width in bearing: $b_{eA} := 2 \cdot \left(R - e_{dA}\right)$ $\quad b_{eA} = 36.3\,ft$

Ellipse soil length in bearing: $l_{eA} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \dfrac{b_{eA}}{2 \cdot R}\right)^2}$ $\quad l_{eA} = 56.1\,ft$

Effective soil length in bearing: $l_{effA} := \sqrt{A_{effA} \cdot \dfrac{l_{eA}}{b_{eA}}}$ $\quad l_{effA} = 47.6\,ft$

Effective soil width in bearing: $b_{effA} := \dfrac{l_{effA}}{l_{eA}} \cdot b_{eA}$ $\quad b_{effA} = 30.8\,ft$

Design bearing pressure: $f_{dA} := \dfrac{V_{dA}}{A_{effA}}$ $\quad f_{dA} = 2367 \cdot psf$

F. Bearing Capacity Check - Extreme Abnormal Loading

Allowable bearing pressure: $f_{allA} := 4200(psf)$ (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure: $\boxed{\dfrac{f_{dA}}{f_{allA}} = 0.56}$

G. Design Soil Bearing Pressure - Earthquake Loading

Design overturning moment: $M_{dEQ} := M_{oEQ}$ $\quad M_{dEQ} = 31808 \cdot ft \cdot k$

Design vertical load: $V_{dEQ} := (1 + E_v) \cdot W_{EQ}$ $\quad V_{dEQ} = 3488 \cdot k$

Design load eccentricity: $e_{dEQ} := \dfrac{M_{dEQ}}{V_{dEQ}}$ $\quad e_{dEQ} = 9.12\, ft$

Circular radius of octagon: $R = 30.75\, ft$

Effective soil area in bearing: $A_{effEQ} := 2 \cdot \left[(R^2) \cdot acos\left(\dfrac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^{\,2}} \right]$ $\quad A_{effEQ} = 1866\, ft^2$

Ellipse soil width in bearing: $b_{eEQ} := 2 \cdot (R - e_{dEQ})$ $\quad b_{eEQ} = 43.3\, ft$

Ellipse soil length in bearing: $l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \dfrac{b_{eEQ}}{2 \cdot R}\right)^2}$ $\quad l_{eEQ} = 58.7\, ft$

Effective soil length in bearing: $l_{effEQ} := \sqrt{A_{effEQ} \cdot \dfrac{l_{eEQ}}{b_{eEQ}}}$ $\quad l_{effEQ} = 50.3\, ft$

Effective soil width in bearing: $b_{effEQ} := \dfrac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$ $\quad b_{effEQ} = 37.1\, ft$

Design bearing pressure: $f_{dEQ} := \dfrac{V_{dEQ}}{A_{effEQ}}$ $\quad f_{dEQ} = 1870 \cdot psf$

H. Bearing Capacity Check - Earthquake Loading

Allowable bearing pressure: $f_{allEQ} := 4200(psf)$ (Reference 2)

Ratio of design bearing pressure to allowable bearing pressure: $\boxed{\dfrac{f_{dEQ}}{f_{allEQ}} = 0.45}$

VI. Foundation Stiffness Evaluation - Single Layer Native Soil Sites

Depth of embedment (half of foundation embedment):	$h := \frac{1}{2} \cdot h_c + h_b$	$h = 39.00 \cdot in$
Area of the footing:	$A = 3133\,ft^2$	
Foundation width:	$D = 61.50\,ft$	
Equivalent circular radius of footing:	$R_{stff} := \sqrt{\frac{A}{\pi}}$	$R_{stff} = 31.6 \cdot ft$
Subsoil density:	$\rho := 136 \cdot \frac{lb}{ft^3}$	(Reference 2)
	$\omega := \rho \cdot g$	$\omega = 136 \cdot pcf$
Design shear wave velocity for interval from 8 to 50 feet:	$V_s := 1636 \cdot \frac{ft}{sec}$	(Reference 2)
Poisson ratio:	$\upsilon := 0.48$	(Reference 2)
Initial shear modulus:	$G_o := \rho \cdot V_s^{\,2}$	$G_o = 11314 \cdot ksf$
Initial elastic modulus:	$E_o := 2 \cdot (1 + \upsilon) \cdot G_o$	$E_o = 33488 \cdot ksf$
Shear modulus ratio:	$\gamma := 1 - (1.0) \cdot \left(\frac{f_{dW}}{3 f_{allW}}\right)^{0.3} = 0.42$	(Reference 9)
Shear modulus:	$G := \gamma \cdot G_o$	$G = 4803 \cdot ksf$
Elastic modulus:	$E := 2 \cdot (1 + \upsilon) \cdot G$	$E = 14218 \cdot ksf$

Rotational embedment Coefficient:	$\eta_{\psi} := 1 + 1.2 \cdot (1 - \upsilon) \cdot \frac{h}{R_{stff}} + 0.2 \cdot (2 - \upsilon) \cdot \left(\frac{h}{R_{stff}}\right)^3$	$\eta_{\psi} = 1.06$	(Reference 5)
Embedment coefficient:	$\eta_x := 1 + 0.55 \cdot (2 - \upsilon) \cdot \frac{h}{R_{stff}}$	$\eta_x = 1.09$	(Reference 5)
Rotational stiffness of soil:	$K_{\psi dyn} := \frac{8 \cdot G \cdot R_{stff}^3}{3 \cdot (1 - \upsilon)} \cdot \eta_{\psi}$	$K_{\psi dyn} = 1120 \cdot \frac{GN \cdot m}{rad}$	(Reference 5)
Required dynamic rotational stiffness:	$K_{\psi req} = 50 \cdot \frac{GN \cdot m}{rad}$		(Reference 3)
Design check:	$\boxed{\frac{K_{\psi dyn}}{K_{\psi req}} = 22.40}$		
Translational stiffness of foundation:	$K_{xdyn} := \frac{32 \cdot (1 - \upsilon) \cdot G \cdot R_{stff}}{7 - 8 \cdot \upsilon} \cdot \eta_x$	$K_{xdyn} = 12660 \cdot \frac{kN}{mm}$	(Reference 8)
Required dynamic translational stiffness:	$K_{xreq} = 1000 \cdot \frac{kN}{mm}$		(Reference 3)
Design check:	$\boxed{\frac{K_{xdyn}}{K_{xreq}} = 12.66}$		

VII. Anchor Bolt Design

A. Strength Reduction and Load Factors

(Reference 1a and Reference 3)

Normal extreme load factor: $\alpha_e = 1.40$

Abnormal extreme load factor: $\alpha_a = 1.14$

Determine controlling extreme load case:

$\alpha_w := \text{if}\left(\alpha_e \cdot M_e > \alpha_a \cdot M_a, \alpha_e, \alpha_a\right)$ $\quad \boxed{\alpha_w = 1.14}$

$M := \text{if}\left(\alpha_e \cdot M_e > \alpha_a \cdot M_a, M_e, M_a\right)$ $\quad M = 41077 \cdot k \cdot ft$

$H := \text{if}\left(\alpha_e \cdot M_e > \alpha_a \cdot M_a, H_e, H_a\right)$ $\quad H = 155 \cdot k$

$W_t := \text{if}\left(\alpha_e \cdot M_e > \alpha_a \cdot M_a, W_{te}, W_{ta}\right)$ $\quad W_t = 590 \cdot k$

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: $\alpha_{d3} := 1.2$

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

Non-Beneficial EQ dead load factor: $\alpha_{d2EQ} := 1.2 + E_v$ $\quad \alpha_{d2EQ} = 1.20$ (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:	$w_{flange} = 10.94 \cdot in$	$w_{flange} = 278 \cdot mm$
Embedment plate hole diameter:	$d_{hl} := 1 \cdot in + \left(\frac{5}{8}\right)(in)$	$d_{hl} = 41.3 \cdot mm$
Embedment ring width:	$w := w_{flange}$	$w = 10.94 \cdot in$
Embedment ring thickness:	$t := 1 \cdot in$	

C. Anchor Bolt Dimensions and Data

Nominal anchor bolt diameter:	$d_b := 1.375 \cdot in$
Bolt area through minimum diameter of threads:	$A_b := 1.56 \cdot in^2$
Washer diameter.:	$d_n := 3 \cdot in$
Outside diamter of PVC bolt sleeve:	$d_{SDR} := 1.9in$
Yield strength:	$F_{yb} := 75 \cdot ksi$
Tensile strength:	$F_t := 100 \cdot ksi$

D. Material Properties

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

E. Anchor Bolt Design

Design loss: $\mu := 20\cdot\%$

Maximum unfactored moment on bolts: $M_{bolt} := \sqrt{\left(\max\left(M_e, M_a\right) + M_{align}\cdot\cos(\Delta)\right)^2 + \left(M_{align}\cdot\sin(\Delta)\right)^2} = 42062\cdot k\cdot ft$

Maximum factored moment on bolts: $M_{ubolt} := \sqrt{\left(\alpha_w\cdot M + \alpha_{d3}\cdot M_{align}\cdot\cos(\Delta)\right)^2 + \left(\alpha_{d3}\cdot M_{align}\cdot\sin(\Delta)\right)^2} = 48009\cdot k\cdot ft$

Maximum factored seismic moment on bolts: $M_{usbolt} := \sqrt{\left(\alpha_{EQ}\cdot M_{OE} + \alpha_{d3}\cdot M_{align}\cdot\cos(\Delta)\right)^2 + \left(\alpha_{d3}\cdot M_{align}\cdot\sin(\Delta)\right)^2} = 30888\cdot k\cdot ft$

Minimum pre-tension for fatigue loading: $T_{preFAT} := \left(\frac{4\cdot\max\left(M_{maxnorth}\right)}{N\cdot D_{avg}} - \frac{W_{mean}}{N}\right)\cdot(1+\mu)$ $\quad T_{preFAT} = 74.8\cdot k$

Set to T_{pre}: $T_{pre} := 75\cdot k = 334\cdot(kN)$

$\%yield := \frac{T_{pre}}{F_{yb}\cdot A_b}$ $\quad \%yield = 64\cdot\%$

$\%ultimate := \frac{T_{pre}}{F_t\cdot A_b}$ $\quad \%ultimate = 48\cdot\%$

Wind load maximum factored tension load in anchor: $T_{uW} := \frac{4\cdot M_{ubolt}}{N\cdot D_{avg}} - \frac{\alpha_{d1}\cdot W_t}{N}$ $\quad T_{uW} = 94\cdot k$

Seismic load maximum factored tension load in anchor: $T_{uEQ} := \frac{4\cdot M_{usbolt}}{N\cdot D_{avg}} - \frac{\alpha_{d1EQ}\cdot W_{OE}}{N}$ $\quad T_{uEQ} = 59\cdot k$

Fatigue load maximum tension load in anchor: $T_{uFAT} := \frac{4\cdot\max\left(M_{maxnorth}\right)}{N\cdot D_{avg}} - \frac{W_{mean}}{N}$ $\quad T_{uFAT} = 62\cdot k$

Determine controlling load case: $T_u := \max\left(T_{uW}, T_{uEQ}, T_{uFAT}\right)$ $\quad T_u = 94\cdot kip$

Design tension strength: $\phi T_n := \min\left(\phi_f\cdot F_t\cdot A_b, \phi_b\cdot F_{yb}\cdot A_b\right)$ $\quad \phi T_n = 105\cdot k$ (Reference 1c)

Design check: $\boxed{\frac{T_u}{\phi T_n} = 0.89}$ 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: $f_{c3} := 5000 \cdot psi$

28-day grout strength: $f_{c28} := 9500 \cdot psi$

Grout thickness: $t_{gr} := 2in$

Bearing area of base flange:

$$A_{br1} := \frac{\pi}{4} \cdot \left(OD^2 - ID^2\right) - N \cdot \frac{\pi \cdot d_{hl}^2}{4} \qquad A_{br1} = 5501 \cdot in^2$$

Section modulus of base flange:

$$S_1 := \frac{\pi}{32 \cdot OD}\left(OD^4 - ID^4\right) \dots + \left[\frac{2}{D_o} \cdot \left[N \cdot \frac{\pi}{64} \cdot d_{hl}^4 + \frac{\pi}{2} \cdot d_{hl}^2 \cdot \left[\sum_{\lambda = 1}^{\frac{N}{4}} \left[\left[\frac{D_i}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2 \dots + \left[\frac{D_o}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2\right]\right]\right]\right]$$

$$S_1 = 218102 \cdot in^3$$

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

Area of limiting bearing within grout: $A_2 := w_{flange} + 2 \cdot t_{gr}$ $\qquad A_2 = 14.9 \cdot in$

$$A := \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right) \qquad A = 1.17$$

B. Check 3 Day Grout Strength

Design bearing strength: $\phi b_{n3} := \phi_{br} \cdot 0.85 \cdot f_{c3} \cdot A$ (Reference 1a) $\phi b_{n3} = 3.2 \cdot ksi$

Ultimate self weight bearing stress: $b_{u3D} := \left(\alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$ $b_{u3D} = 2.4 \cdot ksi$

Ultimate seismic bearing stress: $b_{u3EQ} := \left(\alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}} \right)$ $b_{u3EQ} = 2.4 \cdot ksi$

Determine controlling load case and check capacity: $b_{u3} := \max\left(b_{u3D}, b_{u3EQ}\right)$ $b_{u3} = 2.4 \cdot ksi$ $\boxed{\frac{b_{u3}}{\phi b_{n3}} = 0.75}$

C. Check 28 Day Grout Strength

Design bearing strength: $\phi b_{n28} := \phi_{br} \cdot 0.85 \cdot f_{c28} \cdot A$ (Reference 1a) $\phi b_{n28} = 6.1 \cdot ksi$

Ultimate wind bearing stress: $b_{u28W} := \alpha_{d2} \cdot \frac{W_t}{A_{br1}} + \frac{M_{ubolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$ $b_{u28W} = 5.06 \cdot ksi$

Ultimate seismic bearing stress: $b_{u28EQ} := \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br1}} + \alpha_{EQ} \cdot \frac{M_{usbolt}}{S_1} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}$ $b_{u28EQ} = 4.1 \cdot ksi$

Determine controlling load case and check capacity: $b_{u28} := \max\left(b_{u28W}, b_{u28EQ}\right)$ $b_{u28} = 5.1 \cdot ksi$ $\boxed{\frac{b_{u28}}{\phi b_{n28}} = 0.83}$

D. Check Bottom Flange Bearing on Concrete

Grout thickness: $t_g := 2 \cdot in$

Pullout force due to wind: $P_{uW} := 2\left(\frac{4M_{ubolt}}{N \cdot D_{avg}} - \alpha_{d1} \cdot \frac{W_t}{N}\right)$ $P_{uW} = 188 \cdot k$

Pullout force due to seismic: $P_{uEQ} := 2\left(\frac{4 \cdot M_{usbolt}}{N \cdot D_{avg}} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{N}\right)$ $P_{uEQ} = 118 \cdot k$

Determine controlling load case: $P_u := \max(P_{uW}, P_{uEQ})$ $P_u = 188 \cdot k$

Bearing area at bottom of grout: $A_{br2} := \frac{\pi}{4} \cdot \left[\left(OD + 2 \cdot t_g\right)^2 - \left(ID - 2 \cdot t_g\right)^2\right] - N \cdot \frac{\pi \cdot d_{hl}^2}{4}$ $A_{br2} = 7617 \cdot in^2$

Section modulus at bottom of grout:

$$S_2 := \frac{\pi}{32 \cdot (OD + 2 \cdot t_g)}\left[\left(OD + 2 \cdot t_g\right)^4 - \left(ID - 2 \cdot t_g\right)^4\right] \ldots$$
$$+ -\left[\frac{2}{(OD + 2 \cdot t_g)} \cdot \left[N \cdot \frac{\pi}{64} \cdot d_{hl}^4 + \frac{\pi}{2} \cdot d_{hl}^2 \cdot \left[\sum_{\lambda=1}^{\frac{N}{4}} \left[\left[\frac{D_i}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2 \ldots + \left[\frac{D_o}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2\right]\right]\right]\right]$$

$S_2 = 296991 \cdot in^3$

Ultimate wind stress: $b_{uW} := \frac{M_{ubolt}}{S_2} + \alpha_{d2} \cdot \frac{W_t}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}}$ $b_{uW} = 3.7 \cdot ksi$

Ultimate seismic stress: $b_{uEQ} := \frac{M_{usbolt}}{S_2} + \alpha_{d2EQ} \cdot \frac{W_{OE}}{A_{br2}} + \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br2}}$ $b_{uEQ} = 3.0 \cdot ksi$

Determine controlling load case: $b_u := \max(b_{uW}, b_{uEQ})$ $b_u = 3.7 \cdot ksi$

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: $A_1 := w_{flange} + 2 \cdot t_g$ $A_1 = 14.9 \cdot in$

Area of limiting bearing within concrete: $A_2 := A_1 + \left[C - \left(OD + 2 \cdot t_g\right)\right]$ $A_2 = 47.6 \cdot in$

$A := \min\left(\sqrt{\frac{A_2}{A_1}}, 2\right)$ $A = 1.78$

Design bearing strength: $\phi b_n := \phi_{br} \cdot 0.85 \cdot f_{cp} \cdot A$ (Reference 1a) $\phi b_n = 4.9 \cdot ksi$ $\boxed{\frac{b_u}{\phi b_n} = 0.75}$

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

1) Input Parameters - Verts

Size: $Size_{vert} := 8$

Spacing between vertical legs: $s_r := 18 \cdot in$

Reinforcement bar diameter: $d_r := vlookup(Size_{vert}, ACI\_bar\_table, 1)_0 \cdot in$ $\quad d_r = 1.000 \cdot in$

Area of reinforcement: $A_r := vlookup(Size_{vert}, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_r = 0.79 \cdot in^2$

2) Input Parameters - H-bars

Reinforcement bar size: $Size_H := 9$

Spacing between vertical legs: $s_{rH} := 24 \cdot in$

Reinforcement bar diameter: $d_{rH} := vlookup(Size_H, ACI\_bar\_table, 1)_0 \cdot in$ $\quad d_{rH} = 1.128 \cdot in$

Area of reinforcement: $A_{rH} := vlookup(Size_H, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_{rH} = 1.00 \cdot in^2$

Number of reinforcement bars: $n_z := N$ $\quad n_z = 140$

Development length provided: $l_{dhprovided} := h_e - cc_{bot} - 2 \cdot 1.27 \cdot in - 0.5 \cdot d_r + t$ $\quad l_{dhprovided} = 22.0 \cdot in$

Determine location of neutral axis using vertical force equilibrium and then determine moment capacity.

Guess location of neutral axis: $x_{NA} := 46.826 \cdot in$

Force equilibrium: Given $F_c(x_{NA}) = \phi_b \cdot T_{tot}(x_{NA})$

3) Reinforcement

Verts hook length: $l_h := 12 \cdot d_r + \text{if}\left(d_r \leq 1.0 \cdot in, 4 \cdot d_r, \text{if}\left(d_r \geq 1.693 \cdot in, 6 \cdot d_r, 5 \cdot d_r\right)\right)$

$l_h = 16.0 \cdot in$

Specified distance between verts: $l_h := \text{round}\left(\frac{l_h}{in}, 0\right) \cdot in$ $\quad l_h = 16.0 \cdot in$

Outside diameter of reinforcement: $D_{or} := D_{avg} + s_r$ $\quad D_{or} = 15.54\,ft \quad D_{or} = 186.4 \cdot in$

Inside diameter of reinforcement: $D_{ir} := D_{or} - 2 \cdot s_r$ $\quad D_{ir} = 12.54\,ft \quad D_{ir} = 150.4 \cdot in$

Development length required for bar with standard hook: $l_{dhreq} := 0.02 \cdot \frac{f_{yv}}{\sqrt{f_c \cdot psi}} \cdot d_r$ $\quad l_{dhreq} = 17.0 \cdot in$ (Reference 1a)

H-bar hook length: $l_{hH} := 12 \cdot d_{rH} + \text{if}\left(d_{rH} \leq 1.0 \cdot in, 4 \cdot d_{rH}, \text{if}\left(d_{rH} \geq 1.693 \cdot in, 6 \cdot d_{rH}, 5 \cdot d_{rH}\right)\right)$

$l_{hH} = 19.2 \cdot in$

Specified distance between H-bars: $l_{hH} := \text{round}\left(\frac{l_{hH}}{in}, 0\right) \cdot in$ $\quad l_{hH} = 19.0 \cdot in$

Outside diameter of reinforcement: $D_{orH} := D_{avg} + s_{rH}$ $\quad D_{orH} = 16.04\,ft \quad D_{orH} = 192.4 \cdot in$

Inside diameter of reinforcement: $D_{irH} := D_{orH} - 2 \cdot s_{rH}$ $\quad D_{irH} = 12.04\,ft \quad D_{irH} = 144.4 \cdot in$

Development length required for bar with standard hook: $l_{dhreqH} := 0.02 \cdot \frac{f_{yv}}{\sqrt{f_c \cdot psi}} \cdot d_{rH}$ $\quad l_{dhreqH} = 19.1 \cdot in$ (Reference 1a)

Compressive strain in concrete: $\varepsilon_c := 0.003$

(Reference 1a)

Yield strain of reinforcement: $\varepsilon_y := \frac{f_{yv}}{E_s}$ $\qquad \varepsilon_y = 0.0021$

Beta factor:

$$\beta_{1p} := \text{if}\left[f_{cp} \geq 4000\text{psi}, \max\left[0.85 - 0.05\cdot\left(\frac{\frac{f_{cp}}{\text{psi}} - 4000}{1000}\right), 0.65\right], 0.85\right]$$

$\beta_{1p} = 0.80$

Counter for inner and outer bars (consider one half of the section): $\iota := 1, 2 .. \frac{n_z}{4}$

Angle to individual bar pairs: $\beta_\iota := \frac{4\cdot\pi}{n_z}\cdot\left(\iota - \frac{1}{2}\right)$

Distance from extreme compression edge to bars in inner bar diameter: $d_{in_\iota} := \frac{OD}{2} - \frac{D_{ir}}{2}\cdot\cos(\beta_\iota)$ $\qquad d_{inH_\iota} := \frac{OD}{2} - \frac{D_{irH}}{2}\cdot\cos(\beta_\iota)$

Distance from extreme compression edge to bars in outer bar diameter: $d_{out_\iota} := \frac{OD}{2} - \frac{D_{or}}{2}\cdot\cos(\beta_\iota)$ $\qquad d_{outH_\iota} := \frac{OD}{2} - \frac{D_{orH}}{2}\cdot\cos(\beta_\iota)$

Strain for inner bars (neglected if in compression): $\varepsilon_{in}(x_{NA}, \iota) := \max\left[0, \varepsilon_c\cdot\left(\frac{d_{in_\iota}}{x_{NA}} - 1\right)\right]$ $\qquad \varepsilon_{inH}(x_{NA}, \iota) := \max\left[0, \varepsilon_c\cdot\left(\frac{d_{inH_\iota}}{x_{NA}} - 1\right)\right]$

Strain for outer bars (neglected if in compression): $\varepsilon_{out}(x_{NA}, \iota) := \max\left[0, \varepsilon_c\cdot\left(\frac{d_{out_\iota}}{x_{NA}} - 1\right)\right]$ $\qquad \varepsilon_{outH}(x_{NA}, \iota) := \max\left[0, \varepsilon_c\cdot\left(\frac{d_{outH_\iota}}{x_{NA}} - 1\right)\right]$

Distance from centerline to bars in inner bar diameter: $db_{in_\iota} := d_{in_\iota} - \frac{OD}{2}$ $\qquad db_{inH_\iota} := d_{inH_\iota} - \frac{OD}{2}$

Distance from centerline to bars in outer bar diameter: $db_{out_\iota} := d_{out_\iota} - \frac{OD}{2}$ $\qquad db_{outH_\iota} := d_{outH_\iota} - \frac{OD}{2}$

Total tensile force on inner and outer bars:

$$T_{tot}(x_{NA}) := 2\cdot\sum_{bb=1}^{\frac{n_z}{4}}\left(A_r\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreq}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{in}(x_{NA},bb)\right)\right)\ldots$$
$$+2\cdot\sum_{cc=1}^{\frac{n_z}{4}}\left(A_r\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreq}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{out}(x_{NA},cc)\right)\right)\ldots$$
$$+2\cdot\sum_{dd=1}^{\frac{n_z}{4}}\left(A_{rH}\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreqH}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{inH}(x_{NA},dd)\right)\right)\ldots$$
$$+2\cdot\sum_{ee=1}^{\frac{n_z}{4}}\left(A_{rH}\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreqH}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{outH}(x_{NA},ee)\right)\right)$$

$$T_{tot}(x_{NA}) = 9101\cdot kip$$

Sum of moments caused by bars about centerline of pedestal:

$$M_T(x_{NA}) := 2\cdot\sum_{ff=1}^{\frac{n_z}{4}}\left[A_r\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreq}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{in}(x_{NA},ff)\right)\cdot\left(db_{in_{ff}}\right)\right]\ldots$$
$$+2\cdot\sum_{gg=1}^{\frac{n_z}{4}}\left[A_r\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreq}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{out}(x_{NA},gg)\right)\cdot\left(db_{out_{gg}}\right)\right]\ldots$$
$$+2\cdot\sum_{hh=1}^{\frac{n_z}{4}}\left[A_{rH}\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreqH}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{inH}(x_{NA},hh)\right)\cdot\left(db_{in_{hh}}\right)\right]\ldots$$
$$+2\cdot\sum_{ii=1}^{\frac{n_z}{4}}\left[A_{rH}\cdot\min\left(1,\frac{l_{dhprovided}}{l_{dhreqH}}\right)\cdot\min\left(f_{yv},E_s\cdot\varepsilon_{outH}(x_{NA},ii)\right)\cdot\left(db_{outH_{ii}}\right)\right]$$

$$M_T(x_{NA}) = 31880\cdot kip\cdot ft$$

4) Concrete

Depth of compression block: $a_c(x_{NA}) := \beta_{1p} \cdot x_{NA}$ $a_c(x_{NA}) = 37.5 \cdot in$

Area of compression block:

$$A_{comp}(x_{NA}) := \begin{cases} \int_0^{a_c(x_{NA})} \left(2 \cdot \sqrt{y \cdot OD - y^2}\right) dy & \text{if } a_c(x_{NA}) < w_{flange} \\ \int_0^{\frac{OD-ID}{2}} \left(2 \cdot \sqrt{y \cdot OD - y^2}\right) dy \ldots \\ + \int_{\frac{OD-ID}{2}}^{a_c(x_{NA})} \left[2 \cdot \sqrt{y \cdot OD - y^2} - 2 \cdot \sqrt{\left[y - \left(\frac{OD - ID}{2}\right)\right] \cdot ID - \left[y - \left(\frac{OD - ID}{2}\right)\right]^2}\right] dy & \text{otherwise} \end{cases}$$

$A_{comp}(x_{NA}) = 1662 \cdot in^2$

Bearing capacity of concrete: $F_c(x_{NA}) := \max(b_u, \phi b_n) \cdot A_{comp}(x_{NA})$ $F_c(x_{NA}) = 8191 \cdot k$

Centroid of concrete in compression from top of section:

$$x_c(x_{NA}) := \begin{cases} \frac{1}{A_{comp}(x_{NA})} \cdot \int_0^{a_c(x_{NA})} \left(2 \cdot \sqrt{y \cdot OD - y^2}\right) \cdot y \, dy & \text{if } a_c(x_{NA}) < w_{flange} \\ \frac{1}{A_{comp}(x_{NA})} \cdot \int_0^{\frac{OD-ID}{2}} \left(2 \cdot \sqrt{y \cdot OD - y^2}\right) \cdot y \, dy \ldots \\ + \frac{1}{A_{comp}(x_{NA})} \cdot \int_{\frac{OD-ID}{2}}^{a_c(x_{NA})} \left[\begin{array}{l} 2 \cdot \sqrt{y \cdot OD - y^2} \ldots \\ + -2 \cdot \sqrt{\left[y - \left(\frac{OD - ID}{2}\right)\right] \cdot ID - \left[y - \left(\frac{OD - ID}{2}\right)\right]^2} \end{array}\right] \cdot y \, dy & \text{otherwise} \end{cases}$$

$x_c(x_{NA}) = 16.3 \cdot in$

Sum of moments by compression about centerline of pedestal: $M_C(x_{NA}) := F_c(x_{NA}) \cdot \left(\frac{OD}{2} - x_c(x_{NA})\right)$ $M_C(x_{NA}) = 50098 \cdot kip \cdot ft$

5) Equillibrium and Flexural Checks

Must equal zero for force equilibrium: $\text{Equilibrium} := \left|\phi_b \cdot T_{tot}(x_{NA}) - F_c(x_{NA})\right|$ $\quad \text{Equilibrium} = 0 \cdot \text{kip}$

Solve for location of neutral axis: $x_{NA} := \text{Find}(x_{NA})$ $\quad x_{NA} = 46.826 \cdot \text{in}$

Total moment capacity of pedestal: $\phi M_{nped}(x_{NA}) := M_T(x_{NA}) + M_C(x_{NA})$ $\quad \phi M_{nped}(x_{NA}) = 81978 \cdot \text{k} \cdot \text{ft}$

Controlling pullout moment: $M_u := \max(M_{ubolt}, M_{usbolt})$ $\quad M_u = 48009 \cdot \text{k} \cdot \text{ft}$

Ratio of factored moment to flexural capacity: $\boxed{\dfrac{M_u}{\phi M_{nped}(x_{NA})} = 0.59}$

F. Check Bending Strength of Embedment Plate

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

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

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

Ultimate wind stress: $b_{uplateW} := \frac{M_{ubolt}}{S_1} - \alpha_{d1} \cdot \frac{W_t}{A_{br1}}$ $b_{uplateW} = 2.5 \cdot ksi$

Ultimate seismic stress: $b_{uplateEQ} := \frac{M_{usbolt}}{S_1} - \alpha_{d1EQ} \cdot \frac{W_{OE}}{A_{br1}}$ $b_{uplateEQ} = 1.6 \cdot ksi$

Determine controlling load case: $b_{uplate} := \max\left(b_{uplateW}, b_{uplateEQ}, \alpha_{pt} \cdot \frac{T_{pre} \cdot N}{A_{br1}}\right)$ $b_{uplate} = 2.5 \cdot ksi$

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

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

Check circumferential nut to nut bending

$M_{u.e1} := b_{uplate} \cdot \frac{{d_1}^2}{12} = 4.93 \cdot \frac{in \cdot k}{in}$ $\phi M_n := \phi_b \cdot \min\left(F_y \cdot Z_y, 1.6 \cdot F_y \cdot S_y\right) = 8.10 \cdot \frac{in \cdot k}{in}$ (Reference 1c) $\boxed{\frac{M_{u.e1}}{\phi M_n} = 0.61}$

Check circumferential nut to nut bending at splice

$M_{u.e2} := b_{uplate} \cdot \frac{{d_1}^2}{8} = 7.39 \cdot \frac{in \cdot k}{in}$ (Reference 1c) $\boxed{\frac{M_{u.e2}}{\phi M_n} = 0.91}$

Check radial nut to nut bending

$M_{u.e3} := b_{uplate} \cdot \frac{{d_2}^2}{12} = 1.69 \cdot \frac{in \cdot k}{in}$ (Reference 1c) $\boxed{\frac{M_{u.e3}}{\phi M_n} = 0.21}$

Check nut to edge bending

$M_{u.e4} := \frac{b_{uplate}}{2} \cdot {d_3}^2 = 1.43 \cdot \frac{in \cdot k}{in}$ (Reference 1c) $\boxed{\frac{M_{u.e4}}{\phi M_n} = 0.18}$

Check shear rupture of washer through plate

$V_{uj} := \max\left(\alpha_{pt} \cdot T_{pre}, \frac{P_u}{2}\right) = 93.94 \cdot k$ $\phi V_n := \phi_v \cdot \pi \cdot d_n \cdot t \cdot 0.6 \cdot F_u = 245.99 \cdot k$ (Reference 1c) $\boxed{\frac{V_{uj}}{\phi V_n} = 0.38}$

IX-a. Concrete Design - Extreme Loads

A. Design Functions

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

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

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

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

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

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

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

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

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

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

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

B. Design Soil Bearing Pressure Wind Loading

(Reference 8)

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

$M_{dW} = 49870 \cdot k \cdot ft$

Design vertical load: $V_{dW} := \alpha_{d1} \cdot \left(W_p + W_f + W_s - \frac{F_b}{\alpha_{d1}} + W_t\right)$ $\quad V_{dW} = 2953 \cdot k$

Design load eccentricity: $e_{dW} := \frac{M_{dW}}{V_{dW}}$ $\quad e_{dW} = 16.9\,ft$

Circular radius of octagon: $R := \frac{D}{2}$ $\quad R = 30.8\,ft$

Effective soil area in bearing: $A_{effW} := 2 \cdot \left[\left(R^2\right) \cdot acos\left(\frac{e_{dW}}{R}\right) - e_{dW} \cdot \sqrt{R^2 - e_{dW}^2}\right]$ $\quad A_{effW} = 1003 \cdot ft^2$

Ellipse soil width in bearing: $b_{eW} := 2 \cdot \left(R - e_{dW}\right)$ $\quad b_{eW} = 27.7\,ft$

Ellipse soil length in bearing: $l_{eW} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eW}}{2 \cdot R}\right)^2}$ $\quad l_{eW} = 51.4\,ft$

Effective soil length in bearing: $l_{effW} := \sqrt{A_{effW} \cdot \frac{l_{eW}}{b_{eW}}}$ $\quad l_{effW} = 43.1\,ft$

Design bearing pressure: $f_{dW} := \frac{V_{dW}}{A_{effW}}$ $\quad f_{dW} = 2944 \cdot psf$

Effective soil width in bearing: $b_{effW} := \frac{l_{effW}}{l_{eW}} \cdot b_{eW}$ $\quad b_{effW} = 23.3\,ft$

$x_{startW} := \frac{D}{2} - e_{dW} - \frac{b_{effW}}{2}$ $\quad x_{startW} = 2.23\,ft$

C. Design Soil Bearing Pressure Seismic Loading

(Reference 8)

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

$M_{dEQ} = 32520 \cdot ft \cdot k$

Design vertical load: $V_{dEQ} := \alpha_{d1EQ} \cdot \left(W_p + W_f + W_s - \frac{F_b}{\alpha_{d1EQ}} + W_{OE}\right)$ $\quad V_{dEQ} = 2973 \cdot kip$

Design load eccentricity: $e_{dEQ} := \frac{M_{dEQ}}{V_{dEQ}}$ $\quad e_{dEQ} = 10.9\,ft$

Effective soil area in bearing: $A_{effEQ} := 2 \cdot \left[\left(R^2\right) \cdot \operatorname{acos}\left(\frac{e_{dEQ}}{R}\right) - e_{dEQ} \cdot \sqrt{R^2 - e_{dEQ}^2}\right]$ $\quad A_{effEQ} = 1654 \cdot ft^2$

Ellipse soil width in bearing: $b_{eEQ} := 2 \cdot \left(R - e_{dEQ}\right)$ $\quad b_{eEQ} = 39.6\,ft$

Ellipse soil length in bearing: $l_{eEQ} := 2 \cdot R \cdot \sqrt{1 - \left(1 - \frac{b_{eEQ}}{2 \cdot R}\right)^2}$ $\quad l_{eEQ} = 57.5\,ft$

Effective soil length in bearing: $l_{effEQ} := \sqrt{A_{effEQ} \cdot \frac{l_{eEQ}}{b_{eEQ}}}$ $\quad l_{effEQ} = 49.0\,ft$

Design bearing pressure: $f_{dEQ} := \frac{V_{dEQ}}{A_{effEQ}}$ $\quad f_{dEQ} = 1797 \cdot psf$

Effective soil width in bearing: $b_{effEQ} := \frac{l_{effEQ}}{l_{eEQ}} \cdot b_{eEQ}$ $\quad b_{effEQ} = 33.8\,ft$

$x_{startEQ} := \frac{D}{2} - e_{dEQ} - \frac{b_{effEQ}}{2}$ $\quad x_{startEQ} = 2.93\,ft$

D. Structural Calculations

Area of pedestal: $A_{ped} := \pi \cdot \frac{C^2}{4}$ $\qquad A_{ped} = 254 \cdot ft^2$

Equivalent square dimension: $S_{ped} := \sqrt{A_{ped}}$ $\qquad S_{ped} = 16.0\,ft$

Distance to critical section: $x_{face} := \frac{D - S_{ped}}{2}$ $\qquad x_{face} = 22.8\,ft$

Number of section slices to be taken: $n := \mathrm{trunc}\left(\frac{a}{ft} \cdot 2\right)$ $\qquad n = 36$

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

From a to the critical section (x_{face}): $j := n + 1, n + 2 .. n + 5$

Array counter for all slices: $q := 1, 2 .. n + 5$

Plan location of section: $x_i := \frac{i}{2} \cdot ft$ $\qquad x_j := a + \left(x_{face} - a\right) \cdot \frac{j - n}{5}$

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

Depth, d, as a function of distance along the sloped portion of the foundation is:

$$d_i := h_b + \frac{x_i}{a} \cdot h_c - 3.75 \cdot in$$

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

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}

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

$$+ -\alpha_{d1}\cdot\text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) + \text{StaticSoilWedgeWeight}(0\text{pcf}, \gamma_w)$$

Design Shear from a to x_{face}

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

$$+ \int_a^{x_j} -\alpha_{d1}\cdot\left(\begin{array}{l} \text{ConcreteVolume}(y)\cdot\gamma_c \;... \\ +\text{DrySoilVolume}(h_j, y)\,\gamma_{sdbot} \;... \\ +\text{SaturatedSoilVolume}(h_j, y)\cdot\gamma_{ssbot} \;... \\ +\text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \;... \\ +\dfrac{-\text{VariableSoilWedgeWeight}(y, 0\text{pcf}, \gamma_w)}{\alpha_{d1}} \;... \\ +\dfrac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}}\cdot\gamma_w \end{array}\right) dy + \left(\begin{array}{l} -\alpha_{d1}\cdot\text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \;... \\ +\text{StaticSoilWedgeWeight}(0\text{pcf}, \gamma_w) \end{array}\right)$$

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}

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

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

Design Shear from a to x_{face}

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

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

$$+ StaticSoilWedgeWeight(0pcf, \gamma_w)$$

D2. Summary: Stability, Soil Bearing, Soil Stiffness, and Tower Bottom Flange Anchorage

Factor of Safety Against Overturning:	$Output_{Overturning} = 2.44$	> 1.5
Factor of Safety Against Overturning:	$Output_{Overturning2} = 1.46$	> 1.0
Factor of Safety Against Sliding:	$Output_{Sliding} = 8.92$	> 1.5
Extreme Load Bearing Length Ratio:	$Output_{ExtremeBearing} = 0.74$	> 0.5
Normal Load Bearing Area Ratio:	$Output_{NormalBearing} = 0.996$	> 0.994
Bearing Ratio - Normal:	$Output_{SoilBearingN} = 0.53$	< 1.0
Bearing Ratio - Normal Extreme:	$Output_{SoilBearingEN} = 0.48$	< 1.0
Bearing Ratio - Abnormal Extreme:	$Output_{SoilBearingEA} = 0.56$	< 1.0
Bearing Ratio - Earthquake:	$Output_{SoilBearingEQ} = 0.45$	< 1.0
Rotational and Translational Stiffness Checks (> 1.0):	$Output_{RotationalStiffness2} = 22.4$	
	$Output_{TranslationalStiffness2} = 12.7$	
Anchor Bolt Load Ratio:	$Output_{AnchorBolt} = 0.89$	< 1.0
3-Day Grout Bearing Ratio:	$Output_{Grout3Day} = 0.75$	< 1.0
28-Day Grout Bearing Ratio:	$Output_{Grout28Day} = 0.83$	< 1.0
Concrete Bearing Ratio:	$Output_{ConcreteBearing} = 0.75$	< 1.0
Concrete Pullout Ratio:	$Output_{ConcretePullout2} = 0.59$	< 1.0
	$Output_{ConcretePullout2equil} = 1.00$	= 1.0
Steel Embedment Plate Ratio (max):	$Output_{embedplate} = 0.91$	< 1.0
Soil Bearing Pressure - Normal:	$Output_{BearingPressureN} = 1752 \cdot psf$	
Soil Bearing Pressure - Ex. Normal:	$Output_{BearingPressureEN} = 1997 \cdot psf$	
Soil Bearing Pressure - Ex. Abnormal:	$Output_{BearingPressureEA} = 2367 \cdot psf$	
Soil Bearing Pressure - Earthquake:	$Output_{BearingPressureEQ} = 1870 \cdot psf$	

Bottom Reinforcing Design Moments

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

$$M_{ubot1W_i} := \begin{cases} 0 & \text{if } x_i \le x_{startW} \\ \displaystyle\int_{x_{startW}}^{\min(x_i,\, x_{startW}+b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_i - y)\, dy & \text{otherwise} \end{cases} + -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_i \ldots$$

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

$$+ \int_0^{x_i} -\alpha_{d1} \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_i, y)\, \gamma_{sdbot} \ldots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \ldots \\ + \dfrac{-\text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w)}{\alpha_{d1}} \ldots \\ + \dfrac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{pmatrix} \cdot (x_i - y)\, dy$$

$$M_{ubot1W_j} := \int_{x_{startW}}^{\min(x_j,\, x_{startW}+b_{effW})} f_{dW} \cdot l_{effW} \cdot (x_j - y)\, dy + \int_0^{a} -\alpha_{d1} \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \ldots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \ldots \\ + \dfrac{-\text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w)}{\alpha_{d1}} \ldots \\ + \dfrac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{pmatrix} \cdot (x_j - y)\, dy \ldots$$

$$+ -\alpha_{d1} \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdbot}, \gamma_{ssbot}) \cdot x_j + \text{StaticSoilWedgeWeight}(0pcf, \gamma_w) \cdot x_j \ldots$$

$$+ \int_a^{x_j} -\alpha_{d1} \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_j, y) \cdot \gamma_{sdbot} \ldots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{ssbot} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \ldots \\ + \dfrac{-\text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w)}{\alpha_{d1}} \ldots \\ + \dfrac{-\text{BuoyancyWeight}(y)}{\alpha_{d1}} \cdot \gamma_w \end{pmatrix} \cdot (x_j - y)\, dy$$

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

$$M_{ubot1EQ_i} := \left| \begin{array}{l} 0 \quad \text{if } x_i \le x_{startEQ} \quad \ldots \\ \displaystyle\int_{x_{startEQ}}^{\min(x_i,\, x_{startEQ}+b_{effEQ})} f_{dEQ} \cdot l_{effEQ} \cdot (x_i - y)\, dy \quad \text{otherwise} \end{array} \right.$$

$$+ \int_0^{x_i} -\alpha_{d1EQ} \cdot \left(\begin{array}{l} \text{ConcreteVolume}(y) \cdot \gamma_c \ \ldots \\ + \text{DrySoilVolume}(h_i, y)\, \gamma_{sdbot} \ \ldots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{ssbot} \ \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdbot}, \gamma_{ssbot}) \ \ldots \\ + \dfrac{-\text{VariableSoilWedgeWeight}(y, 0pcf, \gamma_w)}{\alpha_{d1EQ}} \ \ldots \\ + \dfrac{-\text{BuoyancyWeight}(y)}{\alpha_{d1EQ}} \cdot \gamma_w \end{array} \right) \cdot (x_i - y)\, dy \ \ldots$$

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

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

Determine controlling load case for bottom moments:

$$M_{ubot1_q} := \max\left(M_{ubot1W_q}, M_{ubot1EQ_q}\right)$$

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:

$$M_{utop1_i} := \int_0^{x_i} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_i, y) \cdot \gamma_{sdtop} \ldots \\ + \text{SaturatedSoilVolume}(h_i, y) \cdot \gamma_{sstop} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdtop}, \gamma_{sstop}) \end{pmatrix} \cdot (x_i - y)\, dy \ldots$$
$$+ \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdtop}, \gamma_{sstop}) \cdot x_i$$

Alternate distance to critical section based on edge of embedment ring:

$$x_{face\_alt} := \frac{D - \sqrt{\pi \cdot \frac{OD^2}{4}}}{2} \qquad x_{face\_alt} = 24.1\ \text{ft}$$

$$x_{alt_j} := a + (x_{face\_alt} - a) \cdot \frac{j - n}{5}$$

$$M_{utop1_j} := \int_0^{a} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_j, y)\, \gamma_{sdtop} \ldots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdtop}, \gamma_{sstop}) \end{pmatrix} \cdot (x_{alt_j} - y)\, dy \ldots$$
$$+ \int_a^{x_{alt_j}} \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \begin{pmatrix} \text{ConcreteVolume}(y) \cdot \gamma_c \ldots \\ + \text{DrySoilVolume}(h_j, y)\, \gamma_{sdtop} \ldots \\ + \text{SaturatedSoilVolume}(h_j, y) \cdot \gamma_{sstop} \ldots \\ + \text{VariableSoilWedgeWeight}(y, \gamma_{sdtop}, \gamma_{sstop}) \end{pmatrix} \cdot (x_{alt_j} - y)\, dy \ldots$$
$$+ \max(\alpha_{d2}, \alpha_{d2EQ}) \cdot \text{StaticSoilWedgeWeight}(\gamma_{sdtop}, \gamma_{sstop}) \cdot x_{alt_j}$$

E. Top and Bottom Reinforcing Selection

Top middle bars:	$Size_{tmb} := 10$	$s_{topm} := 7 \cdot in$
Top outside bars:	$Size_{tob} := 6$	$s_{topo} := 6 \cdot in$
Bottom middle bars:	$Size_{bmb} := 10$	$s_{botm} := 7.5 \cdot in$
Bottom outside bars:	$Size_{bob} := 6$	$s_{boto} := 10 \cdot in$

Distance from centerline that defines middle/outside boundary: $W_m := \frac{C}{2} + 1.5 \cdot \left(h_b + h_c\right)$ $W_m = 17.25\,ft$

F. Bar Cutoff Locations

Distance of top bar cutoff from edge of footing: $cd_{top} := 12 \cdot ft$

Distance of bottom bar cutoff from edge of footing: $cd_{bot} := 11 \cdot ft$

Assign properties using lookup function, depending on bar size.

$di_{topm} := \text{vlookup}(Size_{tmb}, ACI\_bar\_table, 1)_0 \cdot in$ $\quad di_{topm} = 1.270 \cdot in$

$di_{topo} := \text{vlookup}(Size_{tob}, ACI\_bar\_table, 1)_0 \cdot in$ $\quad di_{topo} = 0.750 \cdot in$

$di_{botm} := \text{vlookup}(Size_{bmb}, ACI\_bar\_table, 1)_0 \cdot in$ $\quad di_{botm} = 1.270 \cdot in$

$di_{boto} := \text{vlookup}(Size_{bob}, ACI\_bar\_table, 1)_0 \cdot in$ $\quad di_{boto} = 0.750 \cdot in$

$A_{topm} := \text{vlookup}(Size_{tmb}, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_{topm} = 1.27 \cdot in^2$

$A_{topo} := \text{vlookup}(Size_{tob}, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_{topo} = 0.44 \cdot in^2$

$A_{botm} := \text{vlookup}(Size_{bmb}, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_{botm} = 1.27 \cdot in^2$

$A_{boto} := \text{vlookup}(Size_{bob}, ACI\_bar\_table, 2)_0 \cdot in^2$ $\quad A_{boto} = 0.44 \cdot in^2$

$W_{topm} := \text{vlookup}(Size_{tmb}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft$ $\quad W_{topm} = 4.303 \cdot \frac{lbf}{ft}$

$W_{topo} := \text{vlookup}(Size_{tob}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft$ $\quad W_{topo} = 1.502 \cdot \frac{lbf}{ft}$

$W_{botm} := \text{vlookup}(Size_{bmb}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft$ $\quad W_{botm} = 4.303 \cdot \frac{lbf}{ft}$

$W_{boto} := \text{vlookup}(Size_{bob}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft$ $\quad W_{boto} = 1.502 \cdot \frac{lbf}{ft}$

G. Bottom Reinforcing Development Length Past Critical Section

1) Middle Bars

Rebar yield strength: $f_y = 75000 \cdot psi$

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

Bottom middle layer bar spacing: $s_{botm} = 7.50 \cdot in$

Bottom middle layer bar diameter: $di_{botm} = 1.270 \cdot in$

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

Coating factor: $\beta := 1.0$ uncoated

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

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

Spacing factor: $c := \min\left(\frac{\min\left(\frac{s_{botm}}{2}, cc_{bot} + \frac{di_{botm}}{2}\right)}{di_{botm}}, 2.5\right)$ $c = 2.5$

Tension development length past critical section: $l_{dbotm} := \frac{3 \cdot f_y \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \sqrt{f_c \cdot psi}} \cdot di_{botm}$ $l_{dbotm} = 40 \cdot in$ (Reference 1a)

2) Outside Bars

Bottom outside layer bar spacing: $s_{boto} = 10.00 \cdot in$

Bottom outside layer bar diameter: $di_{boto} = 0.750 \cdot in$

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

Spacing factor: $c := \min\left(\frac{\min\left(\frac{s_{boto}}{2}, cc_{bot} + \frac{di_{boto}}{2}\right)}{di_{boto}}, 2.5\right)$ $c = 2.5$

Tension development length past critical section: $l_{dboto} := \frac{3 \cdot f_y \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \sqrt{f_c \cdot psi}} \cdot di_{boto}$ $l_{dboto} = 19 \cdot in$ (Reference 1a)

H. Top Reinforcing Development Length Past Critical Section

1) Middle Bars

Top middle layer bar spacing: $s_{topm} = 7.00 \cdot in$

Top middle layer bar size: $di_{topm} = 1.270 \cdot in$

Reinforcement location factor: $\alpha := 1.3$ >12 inches of concrete cast below (Reference 1a)

Coating factor: $\beta := 1.0$ uncoated

Reinforcement size factor: $\gamma := \text{if}\left(di_{topm} < 0.875 \cdot in, 0.8, 1.0\right)$ $\gamma = 1.0$

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

Spacing factor:

$$c := \min\left(\frac{\min\left(\frac{s_{topm}}{2}, cc_{top} + \frac{di_{topm}}{2}\right)}{di_{topm}}, 2.5\right) \qquad c = 2.1$$

Tension development length past critical section:

$$l_{dtopm} := \frac{3 \cdot f_y \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \sqrt{f_c \cdot psi}} \cdot di_{topm} \qquad l_{dtopm} = 63 \cdot in$$

(Reference 1a)

2) Outside Bars

Top outside layer bar spacing: $s_{topo} = 6.00 \cdot in$

Top outside layer bar size: $di_{topo} = 0.750 \cdot in$

Reinforcement size factor: $\gamma := \text{if}\left(di_{topo} < 0.875 \cdot in, 0.8, 1.0\right)$ $\gamma = 0.8$

Spacing factor:

$$c := \min\left(\frac{\min\left(\frac{s_{topo}}{2}, cc_{top} + \frac{di_{topo}}{2}\right)}{di_{topo}}, 2.5\right) \qquad c = 2.5$$

Tension development length past critical section:

$$l_{dtopo} := \frac{3 \cdot f_y \cdot \alpha \cdot \beta \cdot \gamma}{40 \cdot \lambda \cdot c \sqrt{f_c \cdot psi}} \cdot di_{topo} \qquad l_{dtopo} = 25 \cdot in$$

I. Calculate Actual Bottom Moment Capacity

Width of footing at section:

$$W_{bot_q} := \mathrm{if}\left(q \le n, B + 2\cdot x_q, D\right)$$

Number of bars within middle section:

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

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

$$s_{b1bar_q} := \mathrm{if}\left(W_m < \frac{W_{bot_q}}{2}, s_{botm}\cdot n_{botm_q} + s_{boto} - W_m, 0.0in\right)$$

Number of bars across bottom of footing at section:

$$n_{bot_q} := \begin{cases} n_{botm_q} + \left[\mathrm{trunc}\left[\frac{0.5\left(W_{bot_q} - 2W_m\right) - s_{b1bar_q} - cc_{top}}{s_{boto}}\right] + 1\right] & \text{if } W_m < \frac{W_{bot_q}}{2} \\ n_{botm_q} & \text{otherwise} \end{cases}$$

Bar counter:

$$ib := 1, 2 .. n_{bot_{n+5}}$$

Distance of bars from centerline across bottom of footing:

$$z_{bot_{ib}} := \mathrm{if}\left[ib \le n_{botm_{n+5}}, ib\cdot s_{botm}, s_{botm}\cdot n_{botm_{n+5}} + s_{boto}\cdot\left(ib - n_{botm_{n+5}}\right)\right]$$

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:

$$d_{barb_{q,ib}} := \begin{cases} \begin{cases} d_{botm_q} - \left(z_{bot_{ib}} - \frac{B}{2}\right)\cdot\frac{h_c}{a} & \text{if } ib \le n_{botm_q} \\ d_{boto_q} - \left(z_{bot_{ib}} - \frac{B}{2}\right)\cdot\frac{h_c}{a} & \text{otherwise} \end{cases} & \text{if } ib\cdot s_{botm} > \frac{B}{2} \\ d_{botm_q} & \text{otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{sbotm_q} := \mathrm{if}\left(x_q \ge cd_{bot}, n_{botm_q}\cdot 2A_{botm}, \frac{1}{2}\cdot n_{botm_q}\cdot 2A_{botm}\right) + A_{botm}$$

Area of steel provided across section at outside section:

$$A_{sboto_q} := \mathrm{if}\left[x_q \ge cd_{bot}, \left(n_{bot_q} - n_{botm_q}\right)\cdot 2\cdot A_{boto}, \frac{1}{2}\cdot\left(n_{bot_q} - n_{botm_q}\right)\cdot 2\cdot A_{boto}\right]$$

Applying ACI minimum reinforcing requirements:

$$\rho_{min} := \max\left(\frac{3\cdot psi^{0.5}\sqrt{f_c}}{f_y}, \frac{200\cdot psi}{f_y}\right) \qquad \rho_{min} = 0.00283$$

Minimum area of steel required at section:

$$A_{sminb_q} := \rho_{min} \cdot \left[B \cdot d_{botm_q} + \frac{W_{bot_q} - B}{2} \cdot \left[d_{botm_q} + \left(h_b - cc_{bot} - di_{boto} \right) \right] \right]$$ (Reference 1a)

Factored moment considering minimum reinforcing requirements at section:

$$M_{ubot_q} := \mathrm{if}\left(A_{sbotm_q} + A_{sboto_q} \le A_{sminb_q}, \max\left(\frac{4}{3} \cdot M_{ubot1_q}, 0 \right), \max\left(M_{ubot1_q}, 0 \right) \right)$$

Footing is separated into strips containing one bar each. depth of compression block for each strip:

$$a_{botm} := \frac{A_{botm} \cdot f_y}{0.85 \cdot f_c \cdot s_{botm}}$$

Depth of compression block for each outside strip:

$$a_{boto} := \frac{A_{boto} \cdot f_y}{0.85 \cdot f_c \cdot s_{boto}}$$

Distance from section to end of bar for continuous bars:

$$l_{bot1_{q,ib}} := \mathrm{if}\left[z_{bot_{ib}} > \frac{B}{2}, \max\left[x_q - \left(z_{bot_{ib}} - \frac{B}{2} \right) - \sqrt{2} \cdot cc_{top}, 0 \right], \max\left(x_q - cc_{top}, 0 \right) \right]$$

Distance from section to end of bar for cutoff bars:

$$l_{bot2_{q,ib}} := \max\left(x_q - cd_{bot}, 0 \right)$$

Selection of appropriate bar end distance for bar in question:

$$l_{bot_{q,ib}} := \mathrm{if}\left(ib \cdot s_{botm} - \frac{B}{2} < cd_{bot} \wedge \frac{ib}{2} \ne \mathrm{trunc}\left(\frac{ib}{2} \right), l_{bot2_{q,ib}}, l_{bot1_{q,ib}} \right)$$

Factored moment capacity at section:

$$\phi M_{nbot_q} := \phi_b \cdot \left[A_{botm} \cdot f_y \cdot \min\left(\frac{l_{bot_{q,1}}}{l_{dbotm}}, 1 \right) \cdot \left(d_{botm_q} - \frac{a_{botm}}{2} \right) + 2 \cdot \sum_{kk=1}^{n_{botm_q}} \left[A_{botm} \cdot f_y \cdot \min\left(\frac{l_{bot_{q,kk}}}{l_{dbotm}}, 1 \right) \cdot \left(d_{barb_{q,kk}} - \frac{a_{botm}}{2} \right) \right] \ldots \right.$$
$$\left. + 2 \cdot \sum_{jj = \left(n_{botm_q} \right) + 1}^{n_{bot_q}} \left[A_{boto} \cdot f_y \cdot \min\left(\frac{l_{bot_{q,jj}}}{l_{dboto}}, 1 \right) \cdot \left(d_{barb_{q,jj}} - \frac{a_{boto}}{2} \right) \right] \right]$$

Check of factored moment vs. moment capacity at each section:

$$check_{bot_q} := \frac{M_{ubot_q}}{\phi M_{nbot_q}}$$

J. Calculate Actual Top Moment Capacity

Width of footing at section:

$$W_{top_q} := \text{if}\left(q \le n, B + 2\cdot x_q, D\right)$$

Number of bars within middle section:

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

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

$$s_{t1bar_q} := \text{if}\left(W_m < \frac{W_{top_q}}{2}, s_{topm}\cdot n_{topm_q} + s_{topo} - W_m, 0.0in\right)$$

Number of bars across top of footing at section:

$$n_{top_q} := \begin{cases} n_{topm_q} + \left[\text{trunc}\left[\frac{0.5\left(W_{top_q} - 2W_m\right) - s_{t1bar_q} - cc_{top}}{s_{topo}}\right] + 1\right] & \text{if } W_m < \frac{W_{top_q}}{2} \\ n_{topm_q} & \text{otherwise} \end{cases}$$

Bar counter:

$$it := 1, 2 .. n_{top_{n+5}}$$

Distance of bars from centerline across top of footing:

$$z_{top_{it}} := \text{if}\left[it \le n_{topm_{n+5}}, it\cdot s_{topm}, s_{topm}\cdot n_{topm_{n+5}} + s_{topo}\cdot\left(it - n_{topm_{n+5}}\right)\right]$$

Depth of footing for top middle steel at point:

$$d_{topm_i} := h_b + \frac{x_i}{a}\cdot h_c - cc_{top} - di_{topm}$$

$$d_{topm_j} := h_b + h_c - cc_{top} - di_{topm}$$

Depth of footing for top outside steel at point:

$$d_{topo_i} := h_b + \frac{x_i}{a}\cdot h_c - cc_{top} - di_{topo}$$

$$d_{topo_j} := h_b + h_c - cc_{top} - di_{topo}$$

Depth of each bar at section:

$$d_{bart_{q,it}} := \begin{cases} \text{if } it\cdot s_{topm} > \frac{B}{2} \\ \quad \begin{cases} d_{topm_q} - \left(z_{top_{it}} - \frac{B}{2}\right)\cdot\frac{h_c}{a} & \text{if } it \le n_{topm_q} \\ d_{topo_q} - \left(z_{top_{it}} - \frac{B}{2}\right)\cdot\frac{h_c}{a} & \text{otherwise} \end{cases} \\ d_{topm_q} \quad \text{otherwise} \end{cases}$$

Area of steel provided across section at middle section:

$$A_{stopm_q} := \text{if}\left(x_q \ge cd_{top}, n_{topm_q}\cdot 2A_{topm}, \frac{1}{2}\cdot n_{topm_q}\cdot 2A_{topm}\right) + A_{topm}$$

Area of steel provided across section at outside section:

$$A_{stopo_q} := \text{if}\left[x_q \ge cd_{top}, \left(n_{top_q} - n_{topm_q}\right)\cdot 2\cdot A_{topo}, \frac{1}{2}\cdot\left(n_{top_q} - n_{topm_q}\right)\cdot 2\cdot A_{topo}\right]$$

Minimum area of steel required at section:

$$A_{smint_q} := \rho_{min}\cdot\left[B\cdot d_{topm_q} + \frac{W_{top_q} - B}{2}\cdot\left[d_{topm_q} + \left(h_b - cc_{bot} - di_{topo}\right)\right]\right]$$ (Reference 1a)

Factored moment considering minimum reinforcing requirements at section:

$$M_{utop_q} := \text{if}\left(A_{stopm_q} + A_{stopo_q} \le A_{smint_q}, \max\left(\frac{4}{3}\cdot M_{utop1_q}, 0\right), \max\left(M_{utop1_q}, 0\right)\right)$$

Distance from section to end of bar for continuous bars:

$$l_{top1_{q,it}} := \text{if}\left[z_{top_{it}} > \frac{B}{2}, \max\left[x_q - \left(z_{top_{it}} - \frac{B}{2}\right) - \sqrt{2}\cdot cc_{top}, 0\right], \max\left(x_q - cc_{top}, 0\right)\right]$$

Distance from section to end of bar of bar for cut off bars:

$$l_{top2_{q,it}} := \max\left(x_q - cd_{top}, 0\right)$$

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:

$$f_{sm_{q,it}} := \text{if}\left(it \le n_{top_q}, A_{topm}\cdot f_y\cdot \min\left(\frac{l_{top_{q,it}}}{l_{dtopm}}, 1\right), 0\right)$$

Force developed in each outside bar including development length of individual bars:

$$f_{so_{q,it}} := \text{if}\left(it \le n_{top_q}, A_{topo}\cdot f_y\cdot \min\left(\frac{l_{top_{q,it}}}{l_{dtopo}}, 1\right), 0\right)$$

Beta factor:

$$\beta_1 := \text{if}\left[f_c \ge 4000\text{psi}, \max\left[0.85 - 0.05\cdot\left(\frac{\frac{f_c}{\text{psi}} - 4000}{1000}\right), 0.65\right], 0.85\right] \quad \beta_1 = 0.80$$

Depth of neutral axis at section:

$$xop_q := \frac{A_{topm}\cdot f_y\cdot \min\left(\frac{x_q - cc_{top}}{l_{dtopm}}, 1\right) + 2\cdot\sum_{kk=1}^{n_{topm_q}} f_{sm_{q,kk}} + 2\cdot\sum_{jj=\left(n_{topm_q}\right)+1}^{n_{top_q}} f_{so_{q,jj}}}{W_{top_q}\,\beta_1\cdot .85\cdot f_c}$$

Factored moment capacity at section:

$$\phi M_{ntop_q} := \phi_b\cdot\left[A_{topm}\cdot f_y\cdot \min\left(\frac{x_q - cc_{top}}{l_{dtopm}}, 1\right)\cdot\left(d_{bart_{q,1}} - xop_q\right) + W_{top_q}\cdot 0.85\cdot\beta_1\cdot f_c\cdot xop_q\cdot\left[xop_q - \left(\frac{xop_q\cdot\beta_1}{2}\right)\right] \dots \\ + 2\cdot\sum_{kk=1}^{n_{topm_q}}\left[f_{sm_{q,kk}}\cdot\left(d_{bart_{q,kk}} - xop_q\right)\right] + 2\cdot\sum_{jj=\left(n_{topm_q}\right)+1}^{n_{top_q}}\left[f_{so_{q,jj}}\cdot\left(d_{bart_{q,jj}} - xop_q\right)\right]\right]$$

Check of factored moment vs. moment capacity at each section:

$$check_{top_q} := \frac{M_{utop_q}}{\phi M_{ntop_q}}$$

Factored moment capacity in middle section at critical section:

$$\phi M_{nbotm_{n+5}} := \phi_b \cdot \left[A_{botm} \cdot f_y \cdot \min\left[\frac{l_{bot_{(n+5),1}}}{l_{dbotm}}, 1 \right] \cdot \left(d_{botm_{n+5}} - \frac{a_{botm}}{2} \right) + 2 \cdot \sum_{kk=1}^{n_{botm_{n+5}}} \left[A_{botm} \cdot f_y \cdot \min\left[\frac{l_{bot_{(n+5),kk}}}{l_{dbotm}}, 1 \right] \cdot \left[d_{barb_{(n+5),kk}} - \frac{a_{botm}}{2} \right] \right] \right]$$

$$\phi M_{nbotm_{n+5}} = 22989 \cdot kip \cdot ft$$

Factored moment capacity in middle section at critical section:

$$\phi M_{ntopm_{n+5}} := \phi_b \cdot \left[A_{topm} \cdot f_y \cdot \min\left(\frac{x_{n+5} - cc_{top}}{l_{dtopm}}, 1 \right) \cdot \left(d_{bart_{n+5,1}} - xop_{n+5} \right) + W_m \cdot 0.85 \cdot \beta_1 \cdot f_c \cdot xop_{n+5} \cdot \left[xop_{n+5} - \left(\frac{xop_{n+5} \cdot \beta_1}{2} \right) \right] \ldots \right.$$
$$\left. + 2 \cdot \sum_{kk=1}^{n_{topm_{n+5}}} \left[f_{sm_{n+5,kk}} \cdot \left(d_{bart_{n+5,kk}} - xop_{n+5} \right) \right] \right]$$

$$\phi M_{ntopm_{n+5}} = 24739 \cdot kip \cdot ft$$

Unbalanced wind moment on joint: $M_{unbalancedW} := M_{dW}$ $\qquad M_{unbalancedW} = 49870 \cdot kip \cdot ft$

Fraction of wind moment carried by flexure: $\gamma_{fW} := \dfrac{\phi M_{nbotm_{n+5}} + \phi M_{ntopm_{n+5}}}{M_{unbalancedW}}$ $\qquad \gamma_{fW} = 0.96$

Unbalanced seismic moment on joint: $M_{unbalancedEQ} := M_{dEQ}$ $\qquad M_{unbalancedEQ} = 32520 \cdot kip \cdot ft$

Fraction of seismic moment carried by flexure: $\gamma_{fEQ} := \dfrac{\phi M_{nbotm_{n+5}} + \phi M_{ntopm_{n+5}}}{M_{unbalancedEQ}}$ $\qquad \gamma_{fEQ} = 1.47$

K. Bottom Moment Capacity Results

$q =$	$x_q =$ (ft)	$W_{bot_q} =$ (ft)	$A_{sboto_q} + A_{sbotm_q} =$ ($\cdot in^2$)	$A_{sminb_q} =$ ($\cdot in^2$)	$M_{ubot_q} =$ ($\cdot k \cdot ft$)	$\phi M_{nbot_q} =$ ($\cdot k \cdot ft$)	$check_{bot_q} =$
1	0.5	26.5	26.7	8.3	0	119	0.00
2	1.0	27.5	27.9	9.9	0	326	0.00
3	1.5	28.5	29.2	11.6	0	622	0.00
4	2.0	29.5	30.5	13.4	0	981	0.00
5	2.5	30.5	31.7	15.2	0	1416	0.00
6	3.0	31.5	31.7	17.0	0	1954	0.00
7	3.5	32.5	33.0	18.9	0	2539	0.00
8	4.0	33.5	34.3	20.8	0	2870	0.00
9	4.5	34.5	35.6	22.8	0	3188	0.00
10	5.0	35.5	36.0	24.9	45	3502	0.01
11	5.5	36.5	36.0	27.0	159	3830	0.04
12	6.0	37.5	36.4	29.2	298	4171	0.07
13	6.5	38.5	36.4	31.4	462	4497	0.10
14	7.0	39.5	36.9	33.6	651	4829	0.13
15	7.5	40.5	36.9	35.9	864	5140	0.17
16	8.0	41.5	37.3	38.3	1468	5448	0.27
17	8.5	42.5	37.8	40.7	1817	5758	0.32
18	9.0	43.5	37.8	43.1	2197	6066	0.36
19	9.5	44.5	38.2	45.6	2609	6379	0.41
20	10.0	45.5	38.2	48.2	3053	6697	0.46
21	10.5	46.5	38.6	50.8	3527	7011	0.50
22	11.0	47.5	76.9	53.5	3025	7330	0.41
23	11.5	48.5	76.9	56.2	3427	8957	0.38
24	12.0	49.5	77.8	58.9	3852	10694	0.36
25	12.5	50.5	77.8	61.8	4299	12544	0.34
26	13.0	51.5	78.7	64.6	4769	14445	0.33
27	13.5	52.5	79.5	67.5	5261	16378	0.32
28	14.0	53.5	79.5	70.5	5775	18401	0.31
29	14.5	54.5	80.4	73.5	6311	20135	0.31
30	15.0	55.5	80.4	76.6	6869	20840	0.33
31	15.5	56.5	81.3	79.7	7448	21546	0.35
32	16.0	57.5	82.2	82.9	10732	22259	0.48
33	16.5	58.5	82.2	86.1	11561	22975	0.50
34	17.0	59.5	83.0	89.4	12417	23695	0.52
35	17.5	60.5	83.0	92.7	13301	24422	0.54
36	18.0	61.5	83.9	96.1	14213	25150	0.57
37	19.0	61.5	83.9	96.2	16050	25214	0.64
38	19.9	61.5	83.9	96.2	17962	25224	0.71
39	20.9	61.5	83.9	96.2	19974	25224	0.79
40	21.8	61.5	83.9	96.2	22086	25224	0.88
41	22.8	61.5	83.9	96.2	24297	25224	0.96

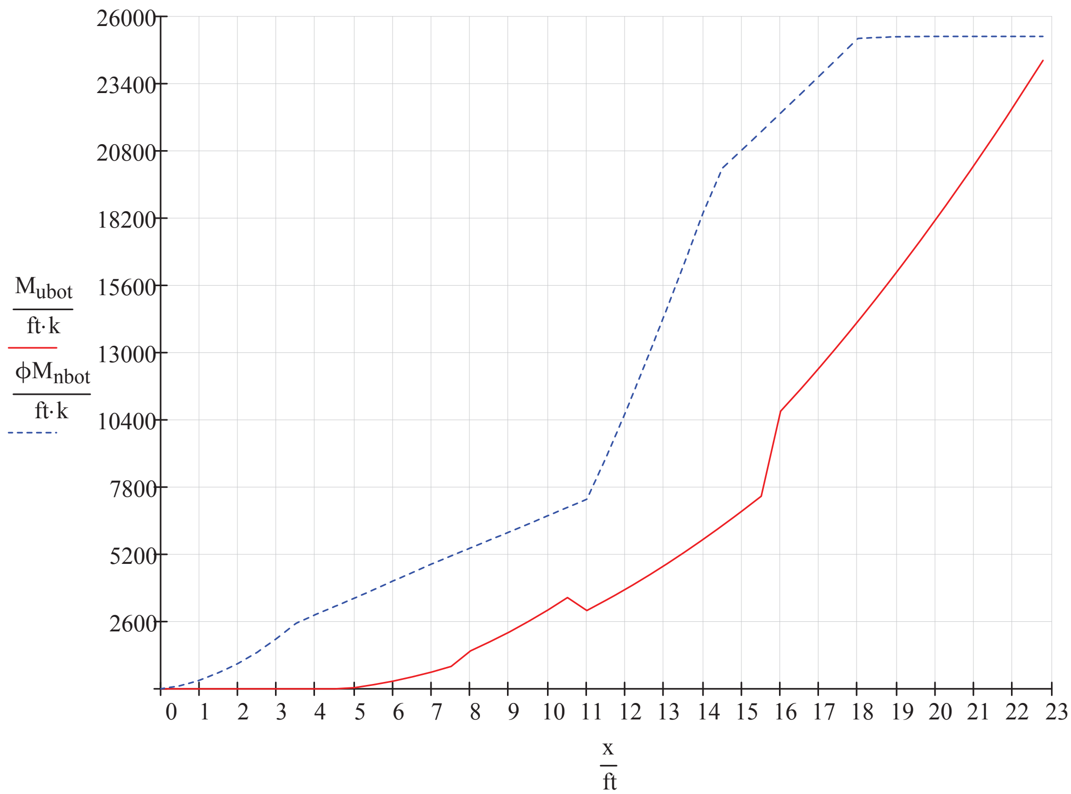
$check_{bot_{n+5}} = 0.96$

L. Top Moment Capacity Results

$q =$	$\frac{x_q}{ft} =$	$\frac{x_{alt_q}}{ft} =$	$\frac{W_{top_q}}{ft} =$	$A_{stopo_q} + A_{stopm_q} =$ ($\cdot in^2$)	$A_{smint_q} =$ ($\cdot in^2$)	$\frac{M_{utop_q}}{k \cdot ft} =$	$\frac{\phi M_{ntop_q}}{k \cdot ft} =$	$check_{top_q} =$
1	0.50	0.0	26.47	29.2	9.2	43	105	0.41
2	1.00	0.0	27.47	30.5	10.8	97	296	0.33
3	1.50	0.0	28.47	31.7	12.5	164	535	0.31
4	2.00	0.0	29.47	31.7	14.3	244	845	0.29
5	2.50	0.0	30.47	33.0	16.1	337	1180	0.29
6	3.00	0.0	31.47	34.3	18.0	443	1592	0.28
7	3.50	0.0	32.47	35.6	19.9	564	2046	0.28
8	4.00	0.0	33.47	36.8	21.8	698	2568	0.27
9	4.50	0.0	34.47	38.1	23.9	848	3146	0.27
10	5.00	0.0	35.47	38.5	25.9	1013	3787	0.27
11	5.50	0.0	36.47	39.0	28.1	1193	4456	0.27
12	6.00	0.0	37.47	39.4	30.2	1390	4839	0.29
13	6.50	0.0	38.47	39.9	32.4	1602	5242	0.31
14	7.00	0.0	39.47	40.3	34.7	1832	5650	0.32
15	7.50	0.0	40.47	40.7	37.0	2079	6048	0.34
16	8.00	0.0	41.47	41.2	39.4	2343	6458	0.36
17	8.50	0.0	42.47	41.6	41.8	3501	6845	0.51
18	9.00	0.0	43.47	42.1	44.3	3902	7245	0.54
19	9.50	0.0	44.47	42.5	46.8	4329	7620	0.57
20	10.00	0.0	45.47	42.9	49.4	4781	7997	0.60
21	10.50	0.0	46.47	43.4	52.0	5260	8374	0.63
22	11.00	0.0	47.47	43.8	54.7	5765	8758	0.66
23	11.50	0.0	48.47	44.3	57.4	6298	9143	0.69
24	12.00	0.0	49.47	88.1	60.2	5144	9548	0.54
25	12.50	0.0	50.47	89.0	63.1	5586	10990	0.51
26	13.00	0.0	51.47	89.9	65.9	6049	12533	0.48
27	13.50	0.0	52.47	90.8	68.9	6535	14162	0.46
28	14.00	0.0	53.47	91.6	71.8	7044	15878	0.44
29	14.50	0.0	54.47	92.5	74.9	7576	17508	0.43
30	15.00	0.0	55.47	93.4	78.0	8131	19180	0.42
31	15.50	0.0	56.47	94.3	81.1	8710	20918	0.42
32	16.00	0.0	57.47	95.2	84.3	9313	22723	0.41
33	16.50	0.0	58.47	96.0	87.5	9942	24595	0.40
34	17.00	0.0	59.47	96.9	90.8	10595	26534	0.40
35	17.50	0.0	60.47	97.8	94.2	11274	28020	0.40
36	18.00	0.0	61.47	98.7	97.5	11979	28894	0.41
37	18.97	19.2	61.50	98.7	97.6	13834	28995	0.48
38	19.92	20.5	61.50	98.7	97.6	15824	29025	0.55
39	20.87	21.7	61.50	98.7	97.6	17970	29026	0.62
40	21.82	22.9	61.50	98.7	97.6	20271	29026	0.70
41	22.77	24.1	61.50	98.7	97.6	22727	29026	0.78

$\boxed{check_{top_{n+5}} = 0.78}$

Factored Moment vs. Moment Capacity - Bottom Reinforcing

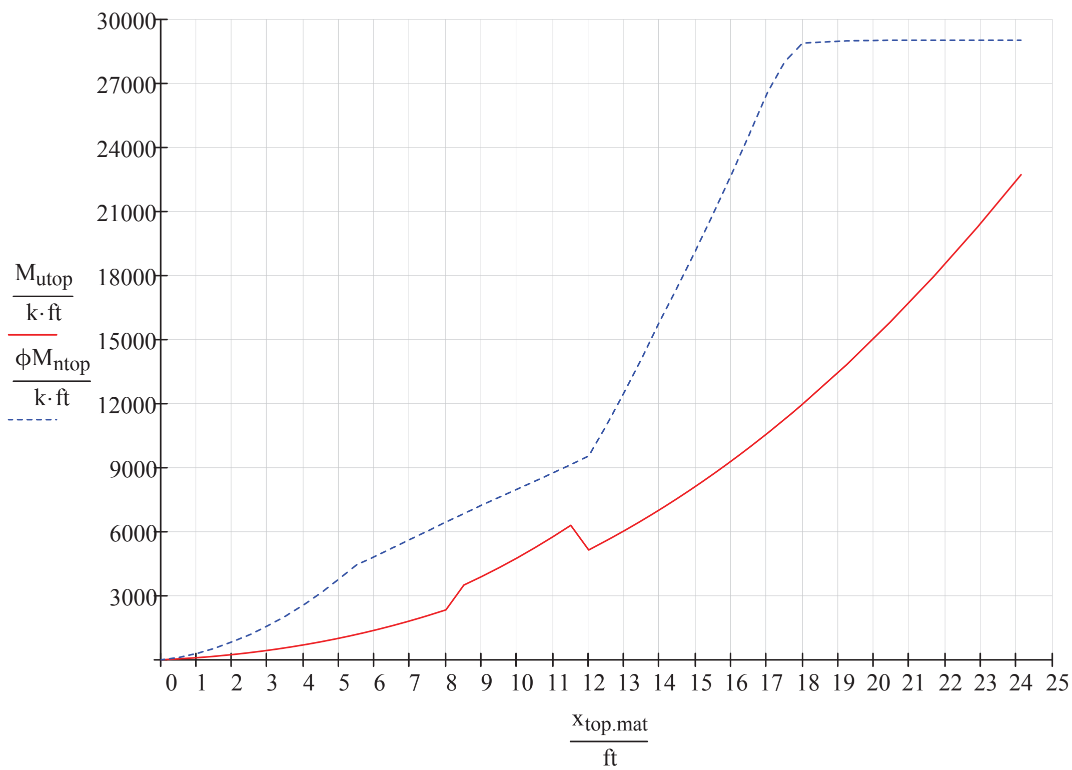


For plotting:

$x_{top.mat_i} := x_i$

$x_{top.mat_j} := x_{alt_j}$

Factored Moment vs. Moment Capacity - Top Reinforcing



M. Check Cutoff Locations

(Reference 1a)

Distance to top bar cutoff location from edge of footing: $cd_{top} = 12.0 \cdot ft$

Counter corresponding to cutoff location: $b := \dfrac{cd_{top}}{x_1}$

Distance to cutoff location from edge of footing: $x_b = 12.0\,ft$

Factored moment capacity at cutoff location: $\phi M_{ntop_b} = 9548 \cdot k \cdot ft$

Footing effective depth at cutoff location: $d_{topm_b} = 3.7\,ft$

Distance equal to effective depth from cutoff location: $cd_{top} + \max\left(d_{topm_b}, 12 \cdot di_{topm}\right) = 15.7\,ft$

Counter corresponding to bin below distance equal to effectiv depth from cutoff location:

$$b_{under} := \mathrm{trunc}\left(2 \cdot \frac{cd_{top} + \max\left(d_{topm_b}, 12 \cdot di_{topm}\right)}{ft}\right) \qquad b_{under} = 31$$

$x_{b_{under}} = 15.5\,ft$

Moment at distance equal to effective depth from cut:

$$M_{utopcutd} := M_{utop_{b_{under}}} \ldots$$
$$+ \left(cd_{top} + \max\left(d_{topm_b}, 12 \cdot di_{topm}\right) - x_{b_{under}}\right) \cdot \left[\frac{\left[M_{utop_{(b_{under}+1)}} - M_{utop_{b_{under}}}\right]}{0.5ft}\right]$$

$M_{utopcutd} = 8982 \cdot k \cdot ft$

Check factored moment at distance equal to effective depth from cutoff location:

$$\boxed{\frac{M_{utopcutd}}{\phi M_{ntop_b}} = 0.94}$$

Distance to bottom bar cutoff location from edge of footing: $cd_{bot} = 11.0 \cdot ft$

Counter corresponding to cutoff location: $b := \dfrac{cd_{bot}}{x_1}$

Distance to cutoff location from edge of footing: $x_b = 11.0\,ft$

Factored moment capacity at cutoff location: $\phi M_{nbot_b} = 7330 \cdot k \cdot ft$

Footing effective depth at cutoff location: $d_{botm_b} = 3.4\,ft$

Distance equal to effective depth from cutoff location: $cd_{bot} + \max\left(d_{botm_b}, di_{botm}\right) = 14.4\,ft$

Counter corresponding to bin below distance equal to effectiv depth from cutoff location: $b_{under2} := \mathrm{trunc}\left(2 \cdot \dfrac{cd_{bot} + \max\left(d_{botm_b}, di_{botm}\right)}{ft}\right)$ $b_{under2} = 28$

$x_{b_{under2}} = 14.0\,ft$

Moment at distance equal to effective depth from cut:

$$M_{ubotcutd} := M_{ubot_{b_{under2}}} \ldots + \left(cd_{bot} + \max\left(d_{botm_b}, di_{botm}\right) - x_{b_{under2}}\right) \cdot \left[\frac{\left[M_{ubot_{\left(b_{under2}+1\right)}} - M_{ubot_{b_{under2}}}\right]}{0.5ft}\right]$$

$M_{ubotcutd} = 6196 \cdot k \cdot ft$

Check factored moment at distance equal to effective depth from cutoff location: $\boxed{\dfrac{M_{ubotcutd}}{\phi M_{nbot_b}} = 0.85}$

IX-b. Moment Capacity of Bottom Reinforcement at 45 degree angle

Distance to critical section from centerline of foundation: $x_o := \frac{\sqrt{\pi}\cdot C}{4}$ $x_o = 95.7\cdot in$

Slant distance on critical section from foundation edge to slope transition point: $a_p := \left(\frac{1}{2}\right)\cdot\sqrt{2\cdot\left(\frac{D-B}{2}\right)^2} - x_o$ $a_p = 57.1\cdot in$

Geometric distance to transition point: $C_1 := \sqrt{2}\,a_p$ $C_1 = 80.8\cdot in$

Height of transition point: $h := h_b + \left(\frac{h_c}{a}\right)\cdot C_1$ $h = 32.2\cdot in$

Number of bars between centerline and critical section: $N_{barsdown} := \mathrm{trunc}\left(\frac{x_o}{\sqrt{2}\cdot s_{botm}}\right)$ $N_{barsdown} = 9$

Spacing to first bar on critical section: $x_1 := \sqrt{2}\cdot s_{botm} - \left(x_o - \sqrt{2}\cdot s_{botm}\cdot N_{barsdown}\right)$ $x_1 = 10.35\cdot in$

Number of middle section bars: $N_{barbm} := n_{botm_{n+5}} - N_{barsdown}$ $N_{barbm} = 18$

Distance to last middle bar: $d_{tp} := \sqrt{2}\cdot\left(N_{barbm} - 1\right)\cdot s_{botm} + x_1$ $d_{tp} = 191\cdot in$

Number of outside section bars: $N_{barbo} := \mathrm{trunc}\left(\frac{\frac{D}{2} - cc_{top} - d_{tp}}{\sqrt{2}\cdot s_{boto}}\right)$ $N_{barbo} = 12$

Number of bars crossing the critical section in diagonal direction 1: $N_{bars1} := N_{barbm} + N_{barbo}$ $N_{bars1} = 30$

Bar counter for diagonal bars in diagonal direction 1: $b_1 := 1, 2 .. N_{bars1}$

Distance from centerline of bars in diagonal direction 1: $x_{b_1} := \mathrm{if}\left[b_1 > N_{barbm}, x_{N_{barbm}} + \sqrt{2}\cdot s_{boto}\cdot\left(b_1 - N_{barbm}\right), x_1 + \sqrt{2}\cdot s_{botm}\cdot\left(b_1 - 1\right)\right]$

Development lengths provided for individual bars: $L_{in_{b_1}} := \min\left[100\cdot in, \sqrt{2}\cdot\left(\frac{D}{2} - cc_{top} - x_{b_1}\right)\right]$ $L_{out_{b_1}} := 100\cdot in$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{bot} - di_{botm} & \text{if } x_{b_1} \le \frac{D}{2} - \left(a_p + 2\cdot x_o\right) \\ \frac{h_b - h}{a_p}\cdot\left[x_{b_1} - \left(\frac{D}{2} - a_p\right)\right] + h - \left(cc_{bot} + di_{botm}\right) & \text{if } x_{b_1} > \frac{D}{2} - a_p \\ \frac{h - \left(h_b + h_c\right)}{2\cdot x_o}\cdot\left[x_{b_1} - \left[\frac{D}{2} - \left(a_p + 2\cdot x_o\right)\right]\right] + h_c + h_b - \left(cc_{bot} + di_{botm}\right) & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \mathrm{trunc}\left[\frac{\left(\frac{D}{2} - cc_{top}\right) - \left(\sqrt{2}\cdot s_{botm} - x_1\right)}{\sqrt{2}\, s_{botm}} + 1\right] \qquad N_{bars2} = 35$$

Average width of tributary. area for individual bars:

$$b := \frac{\frac{D}{2}}{N_{bars1} + N_{bars2}} \qquad b = 5.68\cdot in$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma 1_{b_1} := \begin{cases} \frac{\sqrt{2}\cdot f_y\cdot A_{boto}}{2}\cdot \min\left(\frac{\min\left(L_{in_{b_1}}, L_{out_{b_1}}\right)}{l_{dboto}}, 1\right) & \text{if } b_1 > N_{barbm} \\ \frac{\sqrt{2}\cdot f_y\cdot A_{botm}}{2}\cdot \min\left(\frac{\min\left(L_{in_{b_1}}, L_{out_{b_1}}\right)}{l_{dbotm}}, 1\right) & \text{otherwise} \end{cases}$$

Depth of compression block for bars in diagonal direction 1:

$$a1_{b_1} := \frac{\sigma 1_{b_1}}{0.85\cdot f_c\cdot b}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 .. N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2}\cdot s_{botm}\cdot\left(b_2\right) - 2\cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$Centerbar := N_{barsdown} + 1 \qquad Centerbar = 10$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \frac{Centerbar}{2} > \mathrm{trunc}\left(\frac{Centerbar}{2}\right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$Other_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \qquad Other_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} Other_{bars} & \text{if } \frac{b_2}{2} > \mathrm{trunc}\left(\frac{b_2}{2}\right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b_2}} := 100in$$

$$L_{out2_{b_2}} := \begin{cases} \text{if } C_{bar_{b_2}} = \text{"noncutoff"} \\ \quad \begin{cases} \min\left[100\cdot in, \sqrt{2}\cdot\left(\frac{D}{2} - cc_{top} - \lambda_{b_2}\right)\right] & \text{if } \frac{D}{2} - \lambda_{b_2} < \frac{\sqrt{2}}{4}\cdot(D - B) - x_o \\ \min\left[100\cdot in, \sqrt{2}\cdot\left[\frac{\sqrt{2}}{4}\cdot(D - B) - x_o\right] - cc_{top} + \frac{\sqrt{2}}{2}\cdot\left[\frac{D}{2} - \lambda_{b_2} - \left[\frac{\sqrt{2}}{4}\cdot(D - B) - x_o\right]\right]\right] & \text{otherwise} \end{cases} \\ \max\left[0in, \min\left[100\cdot in, \sqrt{2}\cdot\left[\frac{\sqrt{2}}{4}\cdot(D - B) - x_o\right] - cc_{top} + \frac{\sqrt{2}}{2}\cdot\left[\frac{D}{2} - \lambda_{b_2} - \left[\frac{\sqrt{2}}{4}\cdot(D - B) - x_o\right]\right] - \left(cd_{bot} - cc_{top}\right)\right]\right] & \text{otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b_2} := \begin{cases} h_b + h_c - cc_{bot} - di_{botm} & \text{if } \lambda_{b_2} \le \dfrac{D}{2} - (a_p + 2\cdot x_o) \\ \dfrac{h_b - h}{a_p}\cdot\left[\lambda_{b_2} - \left(\dfrac{D}{2} - a_p\right)\right] + h - (cc_{bot} + di_{botm}) & \text{if } \lambda_{b_2} > \dfrac{D}{2} - a_p \\ \dfrac{h - (h_b + h_c)}{2\cdot x_o}\cdot\left[\lambda_{b_2} - \left[\dfrac{D}{2} - (a_p + 2\cdot x_o)\right]\right] + h_c + h_b - (cc_{bot} + di_{botm}) & \text{otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma 2_{b_2} := \frac{\sqrt{2}\cdot f_y\cdot A_{botm}}{2}\cdot\min\left(\frac{\min\left(L_{in2_{b_2}}, L_{out2_{b_2}}\right)}{l_{dbotm}}, 1\right)$$

Depth of compression block for bars in diagonal direction 2:

$$a2_{b_2} := \frac{\sigma 2_{b_2}}{0.85\cdot f_c\cdot b}$$

Factored moment capacity at section:

$$\phi M_{nbot} := \phi_b\cdot\left[2\cdot\left[\sum_{kk=1}^{N_{bars1}}\left[\left[\sigma 1_{kk}\cdot\left(d_{kk} - \frac{a1_{kk}}{2}\right)\right]\right]\right] \dots + 2\cdot\sum_{kk=1}^{N_{bars2}}\left[\left[\sigma 2_{kk}\cdot\left(d2_{kk} - \frac{a2_{kk}}{2}\right)\right]\right]\right] \qquad \phi M_{nbot} = 25561\cdot k\cdot ft$$

Ultimate moment in bottom reinforcement at critical section: $M_{ubot_{n+5}} = 24297\cdot k\cdot ft$

Check of factored moment against moment capacity at critical section:

$$check_{bot} := \frac{M_{ubot_{n+5}}}{\phi M_{nbot}} \qquad \boxed{check_{bot} = 0.95}$$

IX-c. Moment Capacity of Top Reinforcement at 45 degree angle

Vertical distance to the centerline of top bars in outer section:

$$K_1 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h - h_b}{a_p}\right)\right)} \qquad K_1 = 3.51 \cdot in$$

Vertical distance to the centerline of top bars in middle section:

$$K_2 := \frac{cc_{top} + di_{topm}}{\cos\left(\tan\left(\frac{h_b + h_c - h}{2 \cdot x_o}\right)\right)} \qquad K_2 = 3.32 \cdot in$$

Number of bars between centerline and critical section:

$$N_{barsdown} := \text{trunc}\left(\frac{x_o}{\sqrt{2} \cdot s_{topm}}\right) \qquad N_{barsdown} = 9$$

Spacing to first bar on critical section:

$$x_1 := \sqrt{2} \cdot s_{topm} - \left(x_o - \sqrt{2} \cdot s_{topm} \cdot N_{barsdown}\right) \qquad x_1 = 3.28 \cdot in$$

Number of middle section bars:

$$N_{bartm} := n_{topm_{n+5}} - N_{barsdown} \qquad N_{bartm} = 20$$

Distance to last middle bar:

$$d_{tp} := \sqrt{2} \cdot \left(N_{bartm} - 1\right) \cdot s_{topm} + x_1 \qquad d_{tp} = 191 \cdot in$$

Number of outside section bars:

$$N_{barto} := \text{trunc}\left(\frac{\frac{D}{2} - cc_{top} - d_{tp}}{\sqrt{2} \cdot s_{topo}}\right) \qquad N_{barbo} = 12$$

Number of bars crossing the critical section in diagonal direction 1:

$$N_{bars1} := N_{bartm} + N_{barto} \qquad N_{bars1} = 40$$

Bar counter for diagonal bars in diagonal direction 1:

$$b_1 := 1, 2 .. N_{bars1}$$

Distance from centerline of bars in diagonal direction 1:

$$x_{b_1} := \text{if}\left[b_1 > N_{bartm}, x_{N_{bartm}} + \sqrt{2} \cdot s_{topo} \cdot \left(b_1 - N_{bartm}\right), x_1 + \sqrt{2} \cdot s_{topm} \cdot \left(b_1 - 1\right)\right]$$

Development lengths provided for individual bars:

$$L_{in_{b_1}} := \min\left[100 \cdot in, \sqrt{2} \cdot \left(\frac{D}{2} - cc_{top} - x_{b_1}\right)\right] \qquad L_{out_{b_1}} := 100 \cdot in$$

Depth of individual bars in diagonal direction 1:

$$d_{b_1} := \begin{cases} h_b + h_c - cc_{top} - di_{topm} & \text{if } x_{b_1} \le \frac{D}{2} - \left(a_p + 2 \cdot x_o\right) \\ \frac{h_b - h}{a_p} \cdot \left[x_{b_1} - \left(\frac{D}{2} - a_p\right)\right] + h - K_1 & \text{if } x_{b_1} > \frac{D}{2} - a_p \\ \frac{h - \left(h_b + h_c\right)}{2 \cdot x_o} \cdot \left[x_{b_1} - \left[\frac{D}{2} - \left(a_p + 2 \cdot x_o\right)\right]\right] + h_c + h_b - K_2 & \text{otherwise} \end{cases}$$

Number of bars crossing the critical section in diagonal direction 2:

$$N_{bars2} := \text{trunc}\left[\frac{\left(\frac{D}{2} - cc_{top}\right) - \left(\sqrt{2} \cdot s_{topm} - x_1\right)}{\sqrt{2}\, s_{topm}} + 1\right] \qquad N_{bars2} = 37$$

Developed stress for individual bars for diagonal direction 1:

$$\sigma 1_{b_1} := \begin{cases} \dfrac{\sqrt{2}\cdot f_y\cdot A_{topo}}{2}\cdot \min\left(\dfrac{\min\left(L_{in_{b_1}}, L_{out_{b_1}}\right)}{l_{dtopo}}, 1\right) & \text{if } b_1 > N_{bartm} \\ \dfrac{\sqrt{2}\cdot f_y\cdot A_{topm}}{2}\cdot \min\left(\dfrac{\min\left(L_{in_{b_1}}, L_{out_{b_1}}\right)}{l_{dtopm}}, 1\right) & \text{otherwise} \end{cases}$$

Bar counter for bars in diagonal direction 2:

$$b_2 := 1, 2 .. N_{bars2}$$

Distance from centerline for bars in diagonal direction 2:

$$\lambda_{b_2} := x_1 + \sqrt{2}\cdot s_{topm}\cdot\left(b_2\right) - 2\cdot x_1$$

Bar number crossing critical section corresponding to the center line bar:

$$Centerbar := N_{barsdown} + 1 \qquad Centerbar = 10$$

Type of bar that center bar is:

$$CL_{bar} := \begin{cases} \text{"cutoff"} & \text{if } \dfrac{Centerbar}{2} > \text{trunc}\left(\dfrac{Centerbar}{2}\right) \\ \text{"noncutoff"} & \text{otherwise} \end{cases}$$

Types of bars that other bars are:

$$Other_{bars} := \begin{cases} \text{"cutoff"} & \text{if } CL_{bar} = \text{"noncutoff"} \\ \text{"noncutoff"} & \text{otherwise} \end{cases} \qquad Other_{bars} = \text{"cutoff"}$$

Type of bars that all diagonal direction 2 bars are:

$$C_{bar_{b_2}} := \begin{cases} Other_{bars} & \text{if } \dfrac{b_2}{2} > \text{trunc}\left(\dfrac{b_2}{2}\right) \\ CL_{bar} & \text{otherwise} \end{cases}$$

Development lengths provided for individual bars:

$$L_{in2_{b_2}} := 100in$$

$$L_{out2_{b_2}} := \begin{cases} \text{if } C_{bar_{b_2}} = \text{"noncutoff"} \\ \quad \begin{cases} \min\left[100\cdot in, \sqrt{2}\cdot\left(\dfrac{D}{2} - cc_{top} - \lambda_{b_2}\right)\right] & \text{if } \dfrac{D}{2} - \lambda_{b_2} < \dfrac{\sqrt{2}}{4}\cdot(D-B) - x_o \\ \min\left[100\cdot in, \sqrt{2}\cdot\left[\dfrac{\sqrt{2}}{4}\cdot(D-B) - x_o\right] - cc_{top} + \dfrac{\sqrt{2}}{2}\cdot\left[\dfrac{D}{2} - \lambda_{b_2} - \left[\dfrac{\sqrt{2}}{4}\cdot(D-B) - x_o\right]\right]\right] & \text{otherwise} \end{cases} \\ \max\left[0in, \min\left[100\cdot in, \sqrt{2}\cdot\left[\dfrac{\sqrt{2}}{4}\cdot(D-B) - x_o\right] - cc_{top} + \dfrac{\sqrt{2}}{2}\cdot\left[\dfrac{D}{2} - \lambda_{b_2} - \left[\dfrac{\sqrt{2}}{4}\cdot(D-B) - x_o\right]\right] - \left(cd_{top} - cc_{top}\right)\right]\right] & \text{otherwise} \end{cases}$$

Depth of individual bars in diagonal direction 2:

$$d2_{b_2} := \begin{cases} h_b + h_c - cc_{top} - di_{topm} & \text{if } \lambda_{b_2} \le \dfrac{D}{2} - \left(a_p + 2\cdot x_o\right) \\ \dfrac{h_b - h}{a_p}\cdot\left[\lambda_{b_2} - \left(\dfrac{D}{2} - a_p\right)\right] + h - K_1 & \text{if } \lambda_{b_2} > \dfrac{D}{2} - a_p \\ \dfrac{h - \left(h_b + h_c\right)}{2\cdot x_o}\cdot\left[\lambda_{b_2} - \left[\dfrac{D}{2} - \left(a_p + 2\cdot x_o\right)\right]\right] + h_c + h_b - K_2 & \text{otherwise} \end{cases}$$

Developed stress for individual bars in diagonal direction 2:

$$\sigma 2_{b_2} := \frac{\sqrt{2} \cdot f_y \cdot A_{topm}}{2} \cdot \min\left(\frac{\min\left(L_{in2_{b_2}}, L_{out2_{b_2}}\right)}{l_{dtopm}}, 1\right)$$

Footing is separated into strips containing one bar each. Depth of compression block for each strip:

$$a_{top} := \frac{\sum_{ii=1}^{N_{bars1}} \sigma 1_{ii} + \sum_{jj=1}^{N_{bars2}} \sigma 2_{jj}}{0.85 \cdot f_c \cdot \frac{D}{2}}$$

$$a_{top} = 2.27 \cdot in$$

Factored moment capacity at critical section:

$$\phi M_{ntop} := \phi_b \cdot \left[2 \cdot \left[\sum_{kk=1}^{N_{bars1}} \left[\left[\sigma 1_{kk} \cdot \left(d_{kk} - \frac{a_{top}}{2} \right) \right] \right] \right] \ldots + 2 \cdot \sum_{kk=1}^{N_{bars2}} \left[\left[\sigma 2_{kk} \cdot \left(d2_{kk} - \frac{a_{top}}{2} \right) \right] \right] \right]$$

$$\phi M_{ntop} = 27898 \cdot k \cdot ft$$

Ultimate moment in top reinforcement:

$$M_{utop_{n+5}} = 22727 \cdot k \cdot ft$$

Check of factored moment against moment capacity at critical section:

$$check_{top} := \frac{M_{utop_{n+5}}}{\phi M_{ntop}}$$

$$\boxed{check_{top} = 0.81}$$

IX-d. One-Way Shear Capacity Check

Plan location of section: $x_i := \frac{i}{2} \cdot ft$ $\quad x_j := a + (x_{face} - a) \cdot \frac{j-n}{5}$

Depth, d, as a function of distance along the sloped portion of the foundation is:

$$d_i := h_b + \frac{x_i}{a} \cdot h_c - 3.75 \cdot in$$

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

Determine controlling load case: $V_{u_q} := \text{if}\left(\left|V_{uW_q}\right| \geq \left|V_{uEQ_q}\right|, V_{uW_q}, V_{uEQ_q}\right)$

Location of critical section from edge of footing: $x_{critical} := x_{face} - d_{n+5}$ $\quad x_{critical} = 17.6\,ft$ (Reference 1a)

Array counter for all slices up to the critical section: $qcs := 1, 2 .. \text{trunc}\left(\frac{x_{critical}}{0.5ft}\right) + 1$

Shear capacity between edge and a: $\phi V_{n_i} := \phi_v \cdot 2 \cdot psi^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[B \cdot d_i + 2 \cdot x_i \cdot \left(d_i - x_i \cdot \frac{h_c}{2 \cdot a}\right)\right]$ (Reference 1a)

Shear capacity between a and x_{face}: $\phi V_{n_j} := \phi_v \cdot 2 \cdot psi^{\frac{1}{2}} \cdot \sqrt{f_c} \cdot \left[B \cdot d_j + 2 \cdot a \cdot \left(d_j - \frac{h_c}{2}\right)\right]$ (Reference 1a)

$d_a := 0.75 \cdot in$ Aggregate size factor: $s_{e_q} := \frac{1.38 \cdot 0.9 \cdot d_q}{d_a + 0.63 \cdot in}$ (Reference 1e)

Overall tension in reinforcing steel: $Tension_q := \frac{M_{ubot1_q}}{0.9 \cdot d_q} + V_{u_q}$

Longitudinal strain at middepth of member: $\varepsilon_{x_q} := \frac{Tension_q}{\left(A_{sboto_q} + A_{sbotm_q}\right) \cdot 2 \cdot E_s}$

$$s_{x_q} := 12$$

Aggregate size factor: $s_{x_q} := \text{if}\left(d_a \geq 1.0in, \frac{0.75 \cdot d_q}{in}, \frac{1.25 \cdot d_q}{0.65in + d_a}\right)$

Capacity between edge and a: $\phi V_{n2_i} := \phi_v \left(2 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \frac{2.25}{1 + 1500 \cdot \varepsilon_{x_i}} \cdot \frac{50}{38 + s_{x_i}}\right) \cdot \left[B \cdot d_i + 2 \cdot x_i \cdot \left(d_i - x_i \cdot \frac{h_c}{2 \cdot a}\right)\right]$

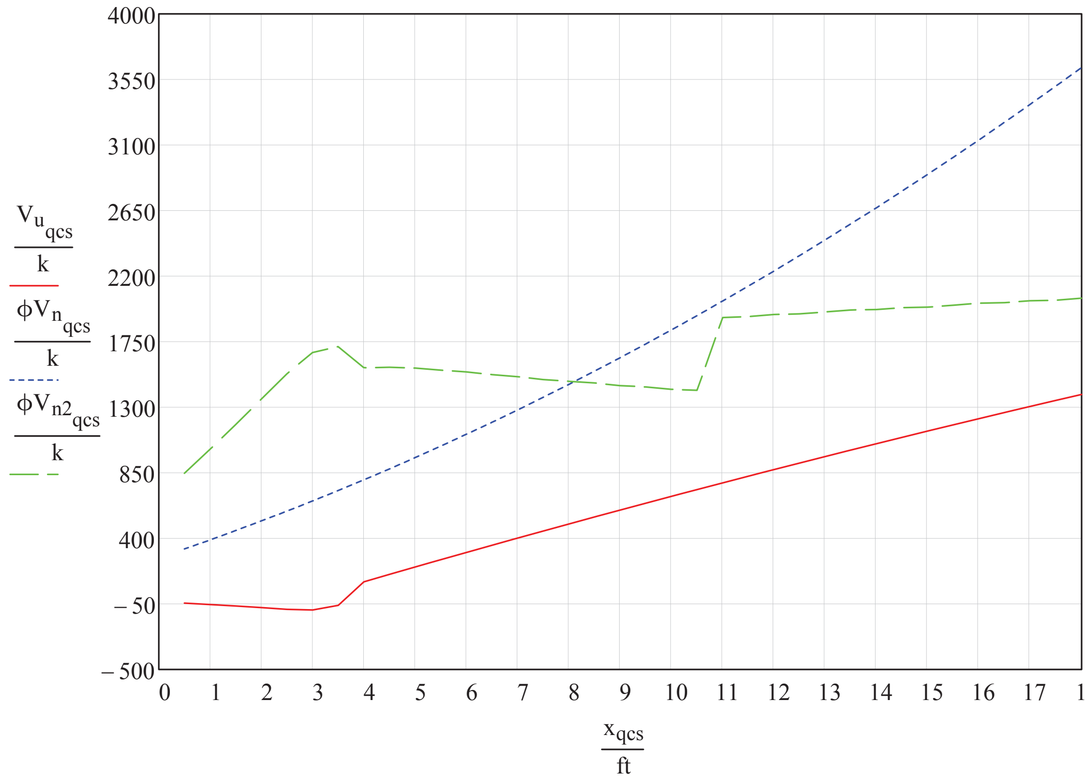
Capacity between a and x_{face_alt}: $\phi V_{n2_j} := \phi_v \cdot \left(2 \cdot \sqrt{\frac{f_c}{psi}} \cdot psi \cdot \frac{2.25}{1 + 1500 \cdot \varepsilon_{x_j}} \cdot \frac{50}{38 + s_{x_j}}\right) \cdot \left[B \cdot d_j + 2 \cdot a \cdot \left(d_j - \frac{h_c}{2}\right)\right]$

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	9.7	-44	328	847	-0.13	-0.05
2	1.0	11.2	-54	390	1012	-0.14	-0.05
3	1.5	12.7	-65	453	1182	-0.14	-0.05
4	2.0	14.2	-76	519	1356	-0.15	-0.06
5	2.5	15.7	-87	587	1529	-0.15	-0.06
6	3.0	17.2	-92	656	1676	-0.14	-0.05
7	3.5	18.7	-60	728	1716	-0.08	-0.03
8	4.0	20.2	102	801	1571	0.13	0.06
9	4.5	21.7	153	877	1574	0.17	0.10
10	5.0	23.2	203	954	1568	0.21	0.13
11	5.5	24.7	253	1033	1555	0.25	0.16
12	6.0	26.2	303	1114	1542	0.27	0.20
13	6.5	27.7	353	1197	1524	0.29	0.23
14	7.0	29.2	402	1282	1510	0.31	0.27
15	7.5	30.7	450	1369	1490	0.33	0.30
16	8.0	32.2	499	1457	1478	0.34	0.34
17	8.5	33.7	547	1548	1466	0.35	0.37
18	9.0	35.2	594	1640	1448	0.36	0.41
19	9.5	36.7	642	1735	1439	0.37	0.45
20	10.0	38.2	689	1831	1423	0.38	0.48
21	10.5	39.7	735	1929	1416	0.38	0.52
22	11.0	41.2	781	2029	1917	0.38	0.41
23	11.5	42.7	827	2131	1923	0.39	0.43
24	12.0	44.2	872	2235	1936	0.39	0.45
25	12.5	45.7	917	2341	1940	0.39	0.47
26	13.0	47.2	962	2449	1954	0.39	0.49
27	13.5	48.7	1006	2559	1967	0.39	0.51
28	14.0	50.2	1050	2670	1971	0.39	0.53
29	14.5	51.7	1094	2784	1984	0.39	0.55
30	15.0	53.2	1137	2899	1987	0.39	0.57
31	15.5	54.7	1180	3016	2001	0.39	0.59
32	16.0	56.2	1222	3136	2014	0.39	0.61
33	16.5	57.7	1264	3257	2018	0.39	0.63
34	17.0	59.2	1306	3380	2032	0.39	0.64
35	17.5	60.7	1347	3505	2034	0.38	0.66
36	18.0	62.2	1388	3631	2049	0.38	0.68

Factored Shear vs. Shear Capacity -

$x_{critical} = 17.6\,ft$



IX-e. Pedestal Two-Way Shear Capacity Check

Effective depth at face of pedestal $d_{mid} := h_b + h_c - cc_{bot} - di_{botm}$ $d_{mid} = 61.7 \cdot in$

Effective depth at face of pedestal $d_{face} := \text{if}\left[\frac{C}{2} + \frac{d_{mid}}{2} < \frac{B}{2}, d_{mid}, h_b + \frac{\frac{D}{2} - \left(\frac{C}{2} + \frac{d_{mid}}{2}\right)}{a} \cdot h_c - cc_{bot} - di_{botm}\right]$

$d_{face} = 61.7 \cdot in$

Area of critical section: $A_c := 2\pi \cdot d_{face} \cdot \left(\frac{C + d_{face}}{2}\right)$ $A_c = 53860 \cdot in^2$

Polar moment of inertia of critical section: $J_c := \pi \cdot d_{face} \cdot \left(\frac{C + d_{face}}{2}\right)^3 + \left(\frac{d_{face}^3}{3}\right)\left(\frac{C + d_{face}}{2}\right)$ $J_c = 5.30 \times 10^8 \cdot in^4$

Perimeter of critical section: $b_0 := 2\pi \cdot \left(\frac{C + d_{face}}{2}\right)$ $b_0 = 873 \cdot in$

Half critical section width: $c := \frac{C + d_{face}}{2}$ (interior column) $c = 139 \cdot in$

Weight of pedestal: $W_p = 191 \cdot kip$

Unfactored vertical wind load on critical section:

$$P_W := W_t + W_p + (h_c + h_b) \cdot \left[\pi \cdot \left(\frac{C + d_{face}}{2}\right)^2\right] \cdot \gamma_c \ldots$$
$$+ \gamma_{sdtop} \cdot [h_s - (h_c + h_b)] \cdot \left[\pi \cdot \left(\frac{C + d_{face}}{2}\right)^2 - \pi \cdot \left(\frac{C}{2}\right)^2\right]$$

$P_W = 1217 \cdot k$

Unfactored vertical seismic load on critical section:

$$P_{EQ} := W_{OE} + W_p + (h_c + h_b) \cdot \left[\pi \cdot \left(\frac{C + d_{face}}{2}\right)^2\right] \cdot \gamma_c \ldots$$
$$+ \gamma_{sdtop} \cdot [h_s - (h_c + h_b)] \cdot \left[\pi \cdot \left(\frac{C + d_{face}}{2}\right)^2 - \pi \cdot \left(\frac{C}{2}\right)^2\right]$$

$P_{EQ} = 1239 \cdot k$

Unbalanced wind moment on joint: $M_{unbalancedW} = 49870 \cdot kip \cdot ft$

Fraction of wind moment that can be carried by flexure: $\gamma_{fW} = 0.96$

Fraction of wind moment carried by shear: $\gamma_{vW} := \max(0.4, 1 - \gamma_{fW}) = 0.40$

Factored shear stress due to wind load at critical section: $v_{uW} := \frac{\alpha_{d2} \cdot P_W}{A_c} + \frac{\gamma_{vW} \cdot M_{unbalancedW} \cdot c}{J_c}$ $v_{uW} = 90 \cdot psi$

Unbalanced seismic moment on joint: $M_{unbalancedEQ} = 32520 \cdot kip \cdot ft$

Fraction of seismic moment that can be carried by flexure: $\gamma_{fEQ} = 1.47$

Fraction of seismic moment carried by shear: $\gamma_{vEQ} := \max(0.4, 1 - \gamma_{fEQ}) = 0.40$

Factored shear stress due to seismic load at critical section:

$$v_{uEQ} := \frac{\alpha_{d2EQ} \cdot P_{EQ}}{A_c} + \frac{\gamma_{vEQ} \cdot M_{unbalancedEQ} \cdot c}{J_c} \qquad v_{uEQ} = 68 \cdot psi$$

Determine controlling load case:

$$v_u := \max(v_{uW}, v_{uEQ}) \qquad v_u = 90\,psi$$

$\beta_c := 1$

$\alpha_s := 40$

$\phi_v = 0.75$

Shear stress capacity:

$$\phi v_c := \phi_v \cdot \min\left[\left(2 + \frac{4}{\beta_c}\right), \left(\frac{\alpha_s \cdot d_{face}}{b_0} + 2\right), 4\right] \cdot \sqrt{f_c \cdot psi} \qquad \phi v_c = 212 \cdot psi$$

(Reference 1a)

Check of factored shear stress vs. shear stress capacity:

$$\boxed{\frac{v_u}{\phi v_c} = 0.42}$$

X. Concrete Design - Fatigue Loads

A. Design Functions

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

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

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

$$\text{StaticSoilWedgeWeightFat}(\gamma_{sd},\gamma_{ss}) := \begin{cases} \gamma_{sd}\cdot\dfrac{B\cdot\tan(\theta_{fat})}{2}\cdot(h_s-h_b)^2 & \text{if } d_{GWTF} \ge h_s-h_b \\ \dfrac{B\cdot\tan(\theta_{fat})}{2}\cdot\left[\gamma_{ss}(h_s-h_b-d_{GWTF})^2+\gamma_{sd}\left[(h_s-h_b)^2-(h_s-h_b-d_{GWTF})^2\right]\right] & \text{otherwise} \end{cases}$$

$$\text{VariableSoilWedgeWeightFat}(y,\gamma_{sd},\gamma_{ss}) := \begin{cases} 0 & \text{if } d_{GWTF} \ge h_s-h_b \\ \text{otherwise} & \begin{cases} \sqrt{2}\cdot\tan(\theta_{fat})\cdot\left[\gamma_{ss}(h_s-h_b-d_{GWTF})^2+\gamma_{sd}\left[(h_s-h_b)^2-(h_s-h_b-d_{GWTF})^2\right]\right] & \text{if } y \le a \\ \tan(\theta_{fat})\cdot\left[\gamma_{ss}(h_s-h_b-d_{GWTF})^2+\gamma_{sd}\left[(h_s-h_b)^2-(h_s-h_b-d_{GWTF})^2\right]\right] & \text{otherwise} \end{cases} \end{cases}$$

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

$$\mathrm{DrySoilVolumeFat}(hj, y) := \begin{cases} \text{if } d_{GWTF} \ge h_s - h_b \\ \quad \begin{cases} \left[\left(h_s - h_b\right) - \dfrac{y}{a} \cdot h_c\right] \cdot (B + 2 \cdot y) + \dfrac{y^2 \cdot h_c}{a} + \sqrt{2} \cdot \tan\left(\theta_{fat}\right) \cdot \left(h_s - h_b\right)^2 & \text{if } y \le a \\ D \cdot \left[\left(h_s - h_b\right) - h_c\right] + h_c \cdot a + \tan\left(\theta_{fat}\right) \cdot \left(h_s - h_b\right)^2 & \text{otherwise} \end{cases} \\ \text{if } d_{GWTF} \le h_s - hj \\ \quad \begin{cases} d_{GWTF} \cdot (B + 2 \cdot y) & \text{if } y \le a \\ D \cdot d_{GWTF} & \text{otherwise} \end{cases} \\ \text{otherwise} \\ \quad \begin{cases} \left[\left(h_s - h_b\right) - \dfrac{y}{a} \cdot h_c\right] \cdot (B + 2 \cdot y) + \left[\dfrac{y^2 \cdot h_c}{a} - \dfrac{a}{h_c} \cdot \left(h_s - h_b - d_{GWTF}\right)^2\right] & \text{if } y \le a \\ \left[D \cdot \left(h_s - h_b - h_c\right) + \left[h_c \cdot a - \dfrac{a}{h_c} \cdot \left(h_s - h_b - d_{GWTF}\right)^2\right]\right] & \text{otherwise} \end{cases} \end{cases}$$

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

$$\mathrm{SaturatedSoilVolumeFat}(hj, y) := \begin{cases} 0 & \text{if } d_{GWTF} \ge h_s - h_b \\ \text{if } d_{GWTF} \le h_s - hj \\ \quad \begin{cases} (B + 2 \cdot y) \cdot \left(h_s - h_b - \dfrac{y}{a} \cdot h_c - d_{GWTF}\right) + \dfrac{y^2 \cdot h_c}{a} & \text{if } y \le a \\ \left(h_s - h_b - h_c - d_{GWTF}\right) \cdot D + h_c \cdot a & \text{otherwise} \end{cases} \\ \dfrac{a}{h_c} \cdot \left(h_s - h_b - d_{GWTF}\right)^2 & \text{otherwise} \end{cases}$$

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

$$\mathrm{BuoyancyWeightFat}(y) := \begin{cases} 0 & \text{if } d_{GWTF} \ge h_s \\ \text{if } d_{GWTF} < h_s \\ \quad \begin{cases} (B + 2 \cdot y) \cdot \left(h_s - d_{GWTF}\right) & \text{if } y \le a \\ \left(h_s - d_{GWTF}\right) \cdot D & \text{otherwise} \end{cases} \end{cases}$$

B. Bottom Reinforcement

(Reference 7)

Depth to reinforcement at critical section for flat portion of footing: $d_{face2} := h_b + h_c - cc_{bot} - 1.5 di_{botm} - 1in$ $\quad d_{face2} = 60.1 \cdot in$

Width of Concrete Resisting Fatigue: $w_f := OD + 3 \cdot (h_c + h_b)$ $\quad w_f = 31.4\,ft$

Cap width to be within "middle" strip: $w_f := \min(w_f, 2 \cdot W_m)$ $\quad w_f = 31.4\,ft$

Area of steel across critical section: $A_s := \frac{w_f}{s_{botm}} \cdot A_{botm}$ $\quad A_s = 63.9 \cdot in^2$

Footing Depth at Edge of Section: $d_{edge} := d_{face2} - \frac{w_f - B}{2} \cdot \frac{h_c}{a} = 51.14 \cdot in$

Concrete Area Resisting Shear: $A_{face} := B \cdot d_{face2} + 2 \cdot \frac{w_f - B}{2} \cdot \left(\frac{d_{edge} + d_{face2}}{2} \right)$ $\quad A_{face} = 22357 \cdot in^2$

C. Fatigue Soil Bearing Pressure

Service load eccentricity: $e_{fNorth_{qr_{north}}} := \frac{MUnique_{north_{qr_{north}}}}{W_{fat}}$

Circular radius of octagon: $R := \frac{D}{2}$ $\quad R = 30.75\,ft$

Effective soil area in bearing:

$$A_{effNorth_{qr_{north}}} := 2 \cdot \left[\left(R^2\right) \cdot acos\left(\frac{e_{fNorth_{qr_{north}}}}{R} \right) \ldots \atop + -e_{fNorth_{qr_{north}}} \cdot \sqrt{R^2 - \left(e_{fNorth_{qr_{north}}}\right)^2} \right]$$

Ellipse soil width in bearing: $b_{eNorth_{qr_{north}}} := 2 \cdot \left(R - e_{fNorth_{qr_{north}}}\right)$

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

Effective soil length in bearing: $l_{effNorth_{qr_{north}}} := \sqrt{A_{effNorth_{qr_{north}}} \cdot \frac{l_{eNorth_{qr_{north}}}}{b_{eNorth_{qr_{north}}}}}$

Effective soil width in bearing: $b_{effNorth_{qr_{north}}} := \dfrac{l_{effNorth_{qr_{north}}}}{l_{eNorth_{qr_{north}}}} \cdot b_{eNorth_{qr_{north}}}$

Maximum fatigue bearing pressure: $f_{fNorth_{qr_{north}}} := \dfrac{W_{fat}}{A_{effNorth_{qr_{north}}}}$

$$x_{startNorth_{qr_{north}}} := \frac{D}{2} - e_{fNorth_{qr_{north}}} - \frac{b_{effNorth_{qr_{north}}}}{2}$$

Foundation plan area: $A_{base} := D^2 - 2\cdot\left(\dfrac{D - B}{2}\right)^2$ $\qquad A_{base} = 3133\,ft^2$

Section modulus of foundation for normal orientation: $S_{normal} := \dfrac{2I_{fdn}}{D}$ $\qquad S_{normal} = 25465\cdot ft^3$

$$W_{fat} = 3160\cdot kip$$

$$\frac{W_{fat}}{A_{base}} = 1009\cdot psf$$

Moment at which the foundation lifts: $M_{maxlift} := \dfrac{W_{fat}}{A_{base}}\cdot S_{normal} = 25682\cdot k\cdot ft$

Maximum soil pressure at point when the foundation lifts: $\sigma_{maxlift} := \dfrac{W_{fat}}{A_{base}} + \dfrac{M_{maxlift}}{S_{normal}} = 2017\cdot psf$

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:

$$\sigma_{north\_max\_soiltrap_{qr_{north}}} := \frac{W_{fat}}{A_{base}} + \frac{MUnique_{north_{qr_{north}}}}{S_{normal}}$$

$$\sigma_{north\_min\_soil_{qr_{north}}} := \begin{array}{|l} \frac{W_{fat}}{A_{base}} - \frac{MUnique_{north_{qr_{north}}}}{S_{normal}} \text{ if } \frac{W_{fat}}{A_{base}} - \frac{MUnique_{north_{qr_{north}}}}{S_{normal}} > 0 \\ \text{"LIFT" otherwise} \end{array}$$

Guess for solver of soil bearing length: $L_b := 52.98ft$

Guess for solver of max soil pressure: $f_{max} := 2633psf$

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:

$$FVALS_{qr_{north}} := W_{fat}$$

$$MTOEVALS_{qr_{north}} := W_{fat} \cdot \frac{D}{2} - MUnique_{north_{qr_{north}}}$$

Given

$$F = \int_0^a (B + 2 \cdot y) \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] dy \ldots + \int_a^{a+B} D \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] dy \ldots + \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] dy$$

$$M_{toe} = \int_0^a (B + 2 \cdot y) \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] y\, dy \ldots + \int_a^{a+B} D \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] y\, dy \ldots + \int_{a+B}^{L_b} [D - 2(a + B - y)] \cdot \left[f_{max} - f_{max} \cdot \left(\frac{y}{L_b} \right) \right] y\, dy$$

$$FUNCTION(F, M_{toe}) := Find\left(\frac{L_b}{ft}, \frac{f_{max}}{psf} \right)$$

Solve the loop for the "LIFT" condition:

$$MapL_{b_{qr_{north}}} := \begin{vmatrix} FUNCTION\left(F_{VALS_{qr_{north}}}, M_{TOEVALS_{qr_{north}}}\right)_0 \cdot ft \ \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"} \\ 0 \ \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} \neq \text{"LIFT"} \end{vmatrix}$$

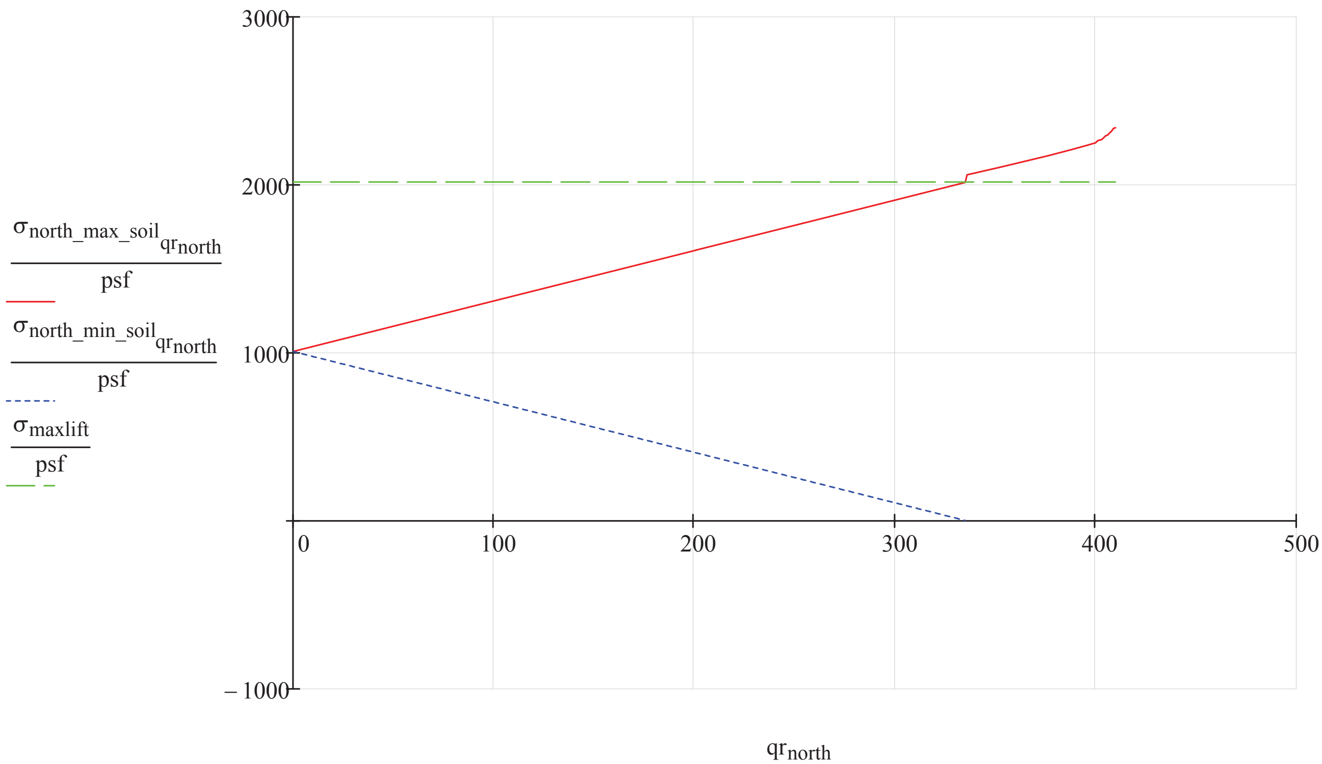
$$\sigma_{north\_max\_soiltri_{qr_{north}}} := \begin{vmatrix} FUNCTION\left(F_{VALS_{qr_{north}}}, M_{TOEVALS_{qr_{north}}}\right)_1 \cdot psf \ \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"} \\ 0 \ \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} \neq \text{"LIFT"} \end{vmatrix}$$

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:

$$\sigma_{north\_max\_soil_{qr_{north}}} := if\left(\sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"}, \sigma_{north\_max\_soiltri_{qr_{north}}}, \sigma_{north\_max\_soiltrap_{qr_{north}}}\right) = ...$$

Soil pressure output for each fatigue load (shown in partial tabular form and graphically):

$qr_{north} =$	$MUnique_{north} =$ (·k·ft)	$\sigma_{north\_min\_soil} =$ (·psf)	$\sigma_{north\_max\_soiltrap} =$ (·psf)	$\sigma_{north\_max\_soiltri} =$ (·psf)	$\sigma_{north\_max\_soil} =$ (·psf)
0	0	1009	1009	0	1009
1	76	1006	1012	0	1012
2	153	1003	1015	0	1015
3	229	1000	1018	0	1018
4	306	997	1021	0	1021
5	382	994	1024	0	1024
6	459	991	1027	0	1027
7	535	988	1030	0	1030
8	612	985	1033	0	1033
9	688	982	1036	0	1036
10	765	979	1039	0	1039
11	841	976	1042	0	1042
12	917	973	1045	0	1045
...	...	...	...	...	...


3000
2000
1000
– 1000
0
100
200
300
400
500
σnorth_max_soil qrnorth / psf
σnorth_min_soil qrnorth / psf
σmaxlift / psf
qrnorth

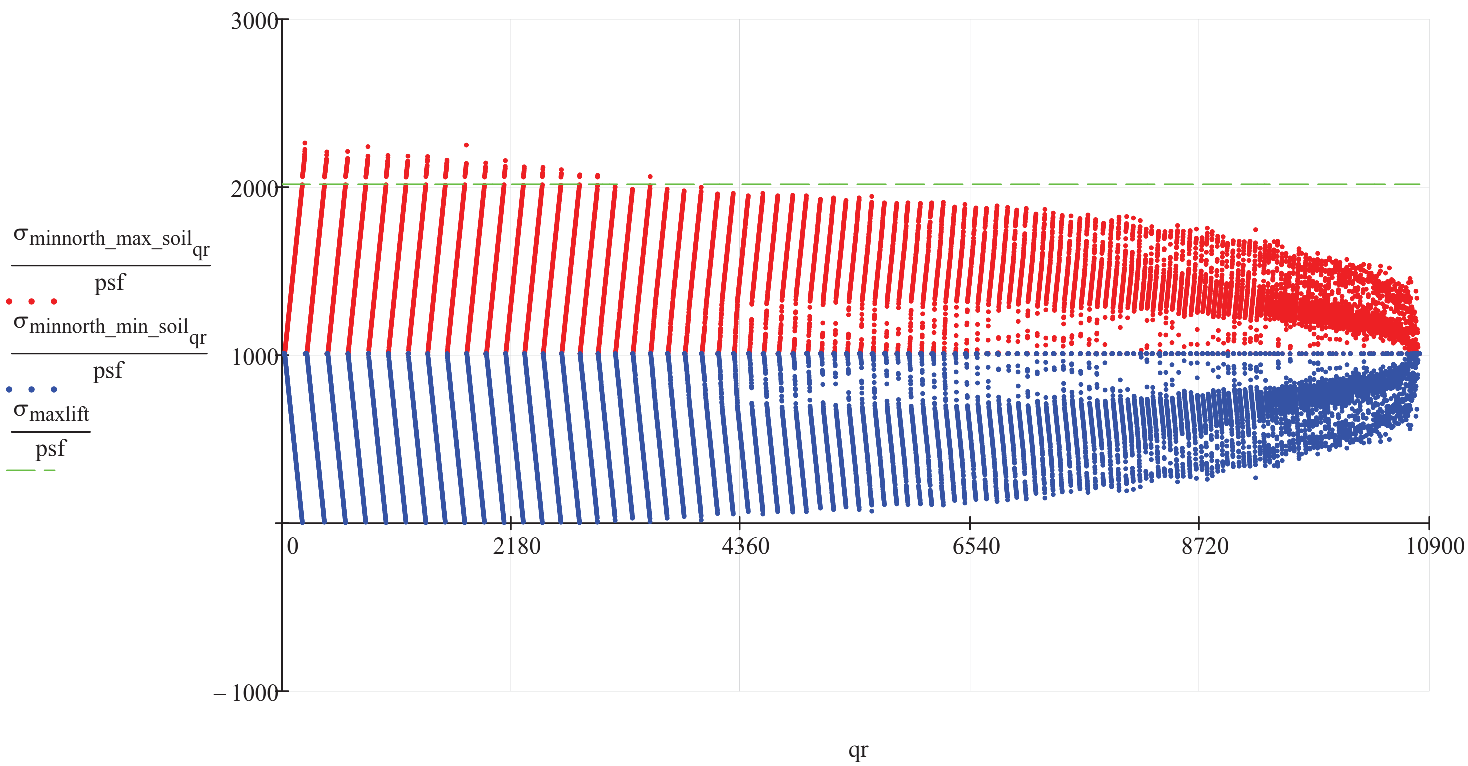
D. Map Unique Matrix Loop Results back to full Minimum Markov or Rain Flow Matrix

$$L_{bmin_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} MapL_{b_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$\sigma_{minnorth\_max\_soil_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} \sigma_{north\_max\_soil_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$Map\sigma_{minnorth\_min\_soil_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} 0 \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"} \\ \text{otherwise} \\ \quad \begin{array}{|l} \sigma_{north\_min\_soil_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \end{array} \right)$$

$$\sigma_{minnorth\_min\_soil_{qr}} := \begin{array}{|l} \text{"LIFT"} \text{ if } Map\sigma_{minnorth\_min\_soil_{qr}} = 0 \\ Map\sigma_{minnorth\_min\_soil_{qr}} \text{ if } Map\sigma_{minnorth\_min\_soil_{qr}} \neq 0 \end{array}$$



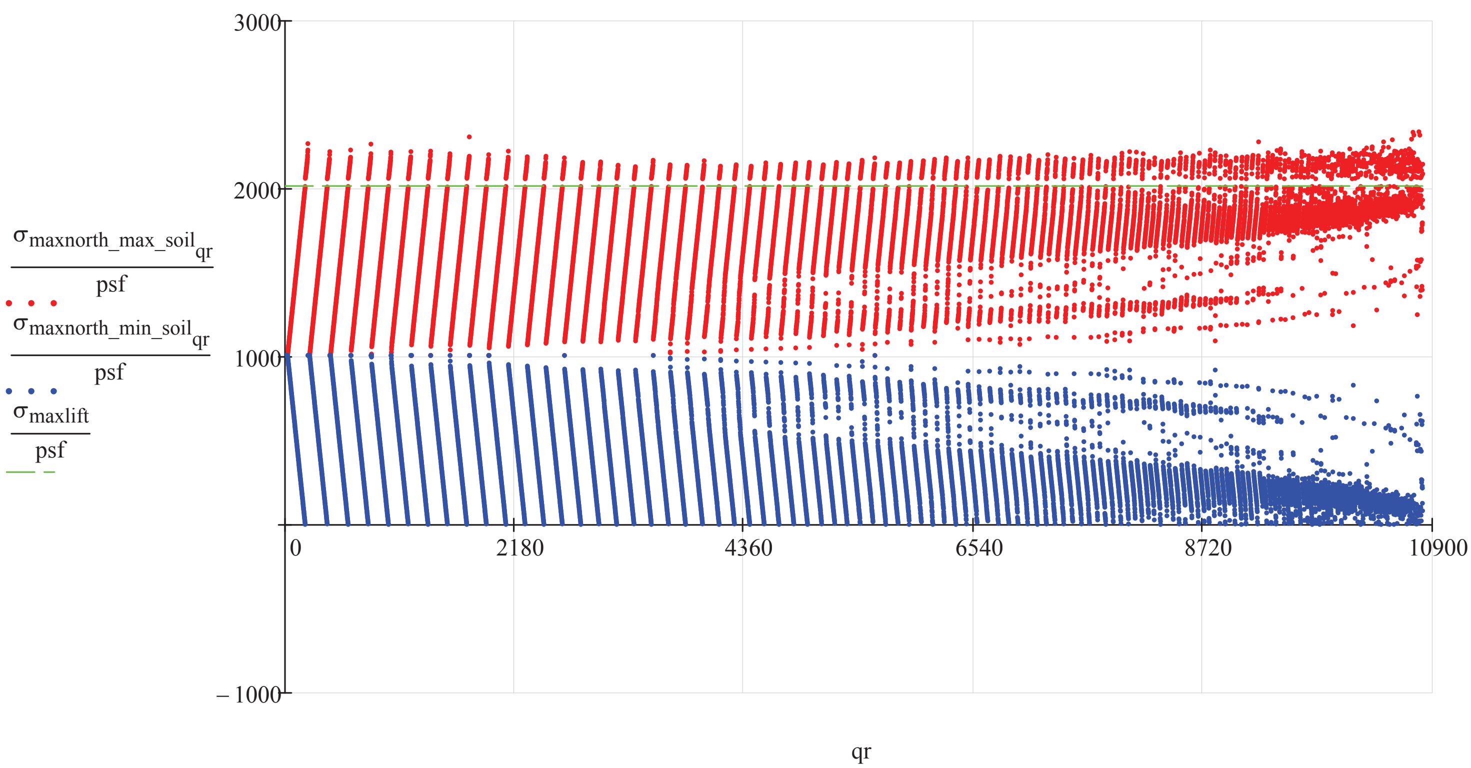
E. Map Unique Matrix Loop Results back to full Maximum Markov or Rain Flow Matrix

$$L_{b_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} MapL_{b_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$\sigma_{maxnorth\_max\_soil_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} \sigma_{north\_max\_soil_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$Map\sigma_{maxnorth\_min\_soil_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} 0 \text{ if } \sigma_{north\_min\_soil_{qr_{north}}} = \text{"LIFT"} \\ \text{otherwise} \\ \quad \begin{array}{|l} \sigma_{north\_min\_soil_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \end{array} \right)$$

$$\sigma_{maxnorth\_min\_soil_{qr}} := \begin{array}{|l} \text{"LIFT"} \text{ if } Map\sigma_{maxnorth\_min\_soil_{qr}} = 0 \\ Map\sigma_{maxnorth\_min\_soil_{qr}} \text{ if } Map\sigma_{maxnorth\_min\_soil_{qr}} \neq 0 \end{array}$$



F. Fatigue Load Bottom Moments and Top Moments at Critical Section

$$M_{fminbotNorth_{qr}} := \begin{cases} \displaystyle\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\left(x_{face}-y\right)\,dy\ \ldots & \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \\ \displaystyle + \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\left(x_{face}-y\right)\,dy\ \ldots & \\ \displaystyle + - \int_0^{x_{face}} \begin{bmatrix} ConcreteVolumeFat(y)\cdot\gamma_c\ \ldots \\ + DrySoilVolumeFat\left(h_b+h_c,y\right)\gamma_{sdbot}\ \ldots \\ + SaturatedSoilVolumeFat\left(h_b+h_c,y\right)\cdot\gamma_{ssbot}\ \ldots \\ + VariableSoilWedgeWeightFat\left[y,\gamma_{sdbot},\left(\gamma_{ssbot}-\gamma_w\right)\right]\ \ldots \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{bmatrix}\cdot\left(x_{face}-y\right)\,dy\ \ldots & \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot},\left(\gamma_{ssbot}-\gamma_w\right)\right]\cdot x_{face} & \\ \displaystyle\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\left(x_{face}-y\right)\,dy\ \ldots & \text{otherwise} \\ \displaystyle + \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\left(x_{face}-y\right)\,dy\ \ldots & \\ \displaystyle + - \int_0^{x_{face}} \begin{bmatrix} ConcreteVolumeFat(y)\cdot\gamma_c\ \ldots \\ + DrySoilVolumeFat\left(h_b+h_c,y\right)\gamma_{sdbot}\ \ldots \\ + SaturatedSoilVolumeFat\left(h_b+h_c,y\right)\cdot\gamma_{ssbot}\ \ldots \\ + VariableSoilWedgeWeightFat\left[y,\gamma_{sdbot},\left(\gamma_{ssbot}-\gamma_w\right)\right]\ \ldots \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{bmatrix}\cdot\left(x_{face}-y\right)\,dy\ \ldots & \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot},\left(\gamma_{ssbot}-\gamma_w\right)\right]\cdot x_{face} & \end{cases}$$

$$M_{fmaxbotNorth_{qr}} := \begin{cases} \displaystyle\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\left(x_{face}-y\right)\,dy \ldots & \text{if } \sigma_{maxnorth\_min\_soil_{qr}} = \text{"LIFT} \\ \displaystyle + \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\left(x_{face}-y\right)\,dy \ldots & \\ \displaystyle + - \int_0^{x_{face}} \begin{bmatrix} \text{ConcreteVolumeFat}(y)\cdot\gamma_c \ldots \\ + \text{DrySoilVolumeFat}\left(h_b + h_c, y\right)\gamma_{sdbot} \ldots \\ + \text{SaturatedSoilVolumeFat}\left(h_b + h_c, y\right)\cdot\gamma_{ssbot} \ldots \\ + \text{VariableSoilWedgeWeightFat}\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -\text{BuoyancyWeightFat}(y)\cdot\gamma_w \end{bmatrix}\cdot\left(x_{face}-y\right)\,dy \ldots & \\ + -\text{StaticSoilWedgeWeightFat}\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot x_{face} & \\ \displaystyle\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\left(x_{face}-y\right)\,dy \ldots & \text{otherwise} \\ \displaystyle + \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\left(x_{face}-y\right)\,dy \ldots & \\ \displaystyle + - \int_0^{x_{face}} \begin{bmatrix} \text{ConcreteVolumeFat}(y)\cdot\gamma_c \ldots \\ + \text{DrySoilVolumeFat}\left(h_b + h_c, y\right)\gamma_{sdbot} \ldots \\ + \text{SaturatedSoilVolumeFat}\left(h_b + h_c, y\right)\cdot\gamma_{ssbot} \ldots \\ + \text{VariableSoilWedgeWeightFat}\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -\text{BuoyancyWeightFat}(y)\cdot\gamma_w \end{bmatrix}\cdot\left(x_{face}-y\right)\,dy \ldots & \\ + -\text{StaticSoilWedgeWeightFat}\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot x_{face} & \end{cases}$$

$$M_{fmintopNorth_{qr}} := \left| \begin{array}{l} \max\left[0k\cdot ft, -\left[\int_a^{D-x_{face\_alt}} \left[\sigma_{minnorth\_max\_soil_{qr}} - \frac{y}{L_{bmin_{qr}}}\cdot\left(\sigma_{minnorth\_max\_soil_{qr}}\right)\right]\cdot D\cdot\left(D - x_{face\_alt} - y\right) dy \ldots \right.\right. \\ \quad + \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\left(D - x_{face\_alt} - y\right) dy \ldots \\ \quad + -\int_0^{D-x_{face\_alt}} \left[\begin{array}{l} ConcreteVolumeFat(y)\cdot\gamma_c \ldots \\ + DrySoilVolumeFat\left(h_b + h_c, y\right)\gamma_{sdbot} \ldots \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right)\cdot\gamma_{ssbot} \ldots \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{array}\right]\cdot\left(D - x_{face\_alt} - y\right) dy \ldots \\ \quad + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot\left(D - x_{face\_alt}\right) \ldots \\ \quad + -W_{mean}\cdot\frac{S_{ped}}{2} \ldots \\ \quad + -\left(\gamma_c\right)\left(h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2} \ldots \\ \quad + -\left(\gamma_c - \gamma_{sdbot}\right)\left(h_p - h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2} \ldots \\ \quad \left.\left. + -M_{minnorth_{qr}}\right]\right] \quad \text{if } \sigma_{m} \\ \max\left[0k\cdot ft, -\left[\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\left(D - x_{face\_alt} - y\right) dy \ldots \right.\right. \\ \quad + \int_a^{D-x_{face\_alt}} \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\left(D - x_{face\_alt} - y\right) dy \ldots \\ \quad + -\int_0^{D-x_{face\_alt}} \left[\begin{array}{l} ConcreteVolumeFat(y)\cdot\gamma_c \ldots \\ + DrySoilVolumeFat\left(h_b + h_c, y\right)\gamma_{sdbot} \ldots \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right)\cdot\gamma_{ssbot} \ldots \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{array}\right]\cdot\left(D - x_{face\_alt} - y\right) dy \ldots \\ \quad + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot\left(D - x_{face\_alt}\right) \ldots \\ \quad + -W_{mean}\cdot\frac{S_{ped}}{2} \ldots \\ \quad + -\left(\gamma_c\right)\left(h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2} \ldots \\ \quad + -\left(\gamma_c - \gamma_{sdbot}\right)\left(h_p - h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2} \ldots \\ \quad \left.\left. + -M_{minnorth_{qr}}\right]\right] \end{array}\right.$$

$$M_{fmaxtopNorth_{qr}} := \left| \begin{array}{l} -\left[\begin{array}{l} \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\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + \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\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + -\int_0^{D-x_{face\_alt}} \left[\begin{array}{l} ConcreteVolumeFat(y)\cdot\gamma_c\ ... \\ + DrySoilVolumeFat\left(h_b + h_c, y\right)\gamma_{sdbot}\ ... \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right)\cdot\gamma_{ssbot}\ ... \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\ ... \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{array}\right]\cdot\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot\left(D - x_{face\_alt}\right)\ ... \\ + -W_{mean}\cdot\frac{S_{ped}}{2}\ ... \\ + -\left(\gamma_c\right)\left(h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2}\ ... \\ + -\left(\gamma_c - \gamma_{sdbot}\right)\left(h_p - h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2}\ ... \\ + -M_{maxnorth_{qr}} \end{array}\right] \text{ if } \sigma_{maxnorth\_min\_s} \\ -\left[\begin{array}{l} \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\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + \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\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + -\int_0^{D-x_{face\_alt}} \left[\begin{array}{l} ConcreteVolumeFat(y)\cdot\gamma_c\ ... \\ + DrySoilVolumeFat\left(h_b + h_c, y\right)\gamma_{sdbot}\ ... \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right)\cdot\gamma_{ssbot}\ ... \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\ ... \\ + -BuoyancyWeightFat(y)\cdot\gamma_w \end{array}\right]\cdot\left(D - x_{face\_alt} - y\right)\,dy\ ... \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\cdot\left(D - x_{face\_alt}\right)\ ... \\ + -W_{mean}\cdot\frac{S_{ped}}{2}\ ... \\ + -\left(\gamma_c\right)\left(h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2}\ ... \\ + -\left(\gamma_c - \gamma_{sdbot}\right)\left(h_p - h_{pe}\right)\cdot\left(\frac{\pi}{4}\cdot C^2\right)\frac{S_{ped}}{2}\ ... \\ + -M_{maxnorth_{qr}} \end{array}\right] \text{ otherw} \end{array}\right.$$

G. Fatigue Load Shear at Critical Section

$$V_{EdminNorth_{qr}} := \max\left[0\text{kip},\ \begin{cases} \displaystyle\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\ \ldots \\ \displaystyle + \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\ \ldots \\ \displaystyle + -\int_0^{x_{face}} \begin{bmatrix} \text{ConcreteVolumeFat}(y)\cdot\gamma_c\ \ldots \\ + \text{DrySoilVolumeFat}\left(h_b + h_c, y\right)\gamma_{sdbot}\ \ldots \\ + \text{SaturatedSoilVolumeFat}\left(h_b + h_c, y\right)\cdot\gamma_{ssbot}\ \ldots \\ + \text{VariableSoilWedgeWeightFat}\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\ \ldots \\ + -\text{BuoyancyWeightFat}(y)\cdot\gamma_w \end{bmatrix} dy\ \ldots \\ + -\text{StaticSoilWedgeWeightFat}\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] & \text{if } \sigma_{minnorth\_min\_soil_{qr}} = \text{"LIFT"} \\ \displaystyle\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\ \ldots \\ \displaystyle + \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\ \ldots \\ \displaystyle + -\int_0^{x_{face}} \begin{bmatrix} \text{ConcreteVolumeFat}(y)\cdot\gamma_c\ \ldots \\ + \text{DrySoilVolumeFat}\left(h_b + h_c, y\right)\gamma_{sdbot}\ \ldots \\ + \text{SaturatedSoilVolumeFat}\left(h_b + h_c, y\right)\cdot\gamma_{ssbot}\ \ldots \\ + \text{VariableSoilWedgeWeightFat}\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right]\ \ldots \\ + -\text{BuoyancyWeightFat}(y)\cdot\gamma_w \end{bmatrix} dy\ \ldots \\ + -\text{StaticSoilWedgeWeightFat}\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] & \text{otherwise} \end{cases}\right]$$

$$V_{EdmaxNorth_{qr}} := \begin{cases} \int_0^a \left[\sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot \left(\sigma_{maxnorth\_max\_soil_{qr}}\right)\right] \cdot (B + 2 \cdot y)\, dy \ldots & \text{if } \sigma_{maxnorth\_min\_soil_{qr}} = \text{"LIFT"} \\ + \int_a^{x_{face}} \left[\sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{L_{b_{qr}}} \cdot \left(\sigma_{maxnorth\_max\_soil_{qr}}\right)\right] \cdot D\, dy \ldots & \\ + -\int_0^{x_{face}} \begin{bmatrix} ConcreteVolumeFat(y) \cdot \gamma_c \ldots \\ + DrySoilVolumeFat\left(h_b + h_c, y\right) \gamma_{sdbot} \ldots \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right) \cdot \gamma_{ssbot} \ldots \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -BuoyancyWeightFat(y) \cdot \gamma_w \end{bmatrix} dy \ldots & \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] & \\ \int_0^a \left[\sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left(\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}\right)\right] \cdot (B + 2 \cdot y)\, dy \ldots & \text{otherwise} \\ + \int_a^{x_{face}} \left[\sigma_{maxnorth\_max\_soil_{qr}} - \frac{y}{D} \cdot \left(\sigma_{maxnorth\_max\_soil_{qr}} - \sigma_{maxnorth\_min\_soil_{qr}}\right)\right] \cdot D\, dy \ldots & \\ + -\int_0^{x_{face}} \begin{bmatrix} ConcreteVolumeFat(y) \cdot \gamma_c \ldots \\ + DrySoilVolumeFat\left(h_b + h_c, y\right) \gamma_{sdbot} \ldots \\ + SaturatedSoilVolumeFat\left(h_b + h_c, y\right) \cdot \gamma_{ssbot} \ldots \\ + VariableSoilWedgeWeightFat\left[y, \gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] \ldots \\ + -BuoyancyWeightFat(y) \cdot \gamma_w \end{bmatrix} dy \ldots & \\ + -StaticSoilWedgeWeightFat\left[\gamma_{sdbot}, \left(\gamma_{ssbot} - \gamma_w\right)\right] & \end{cases}$$

H. Shear and Moment Summary

Results in partial tabular form:

$qr =$	$V_{EdminNorth} =$ (kip)	$V_{EdmaxNorth} =$ (kip)	$M_{fminbotNorth} =$ (k·ft)	$M_{fmaxbotNorth} =$ (k·ft)	$M_{fmintopNorth} =$ (k·ft)	$M_{fmaxtopNorth} =$ (k·ft)
0	237	237	2463	2463	0	-1823
1	237	237	2463	2463	0	-1823
2	237	237	2463	2463	0	-1823
3	237	237	2463	2463	0	-1823
4	237	237	2463	2463	0	-1823
5	237	237	2463	2463	0	-1823
6	237	237	2463	2463	0	-1823
7	237	237	2463	2463	0	-1823
8	237	237	2463	2463	0	-1823
9	237	237	2463	2463	0	-1823
10	237	237	2463	2463	0	-1823
11	237	237	2463	2463	0	-1823
12	237	237	2463	2463	0	-1823
...	...	...	...	...	...	...

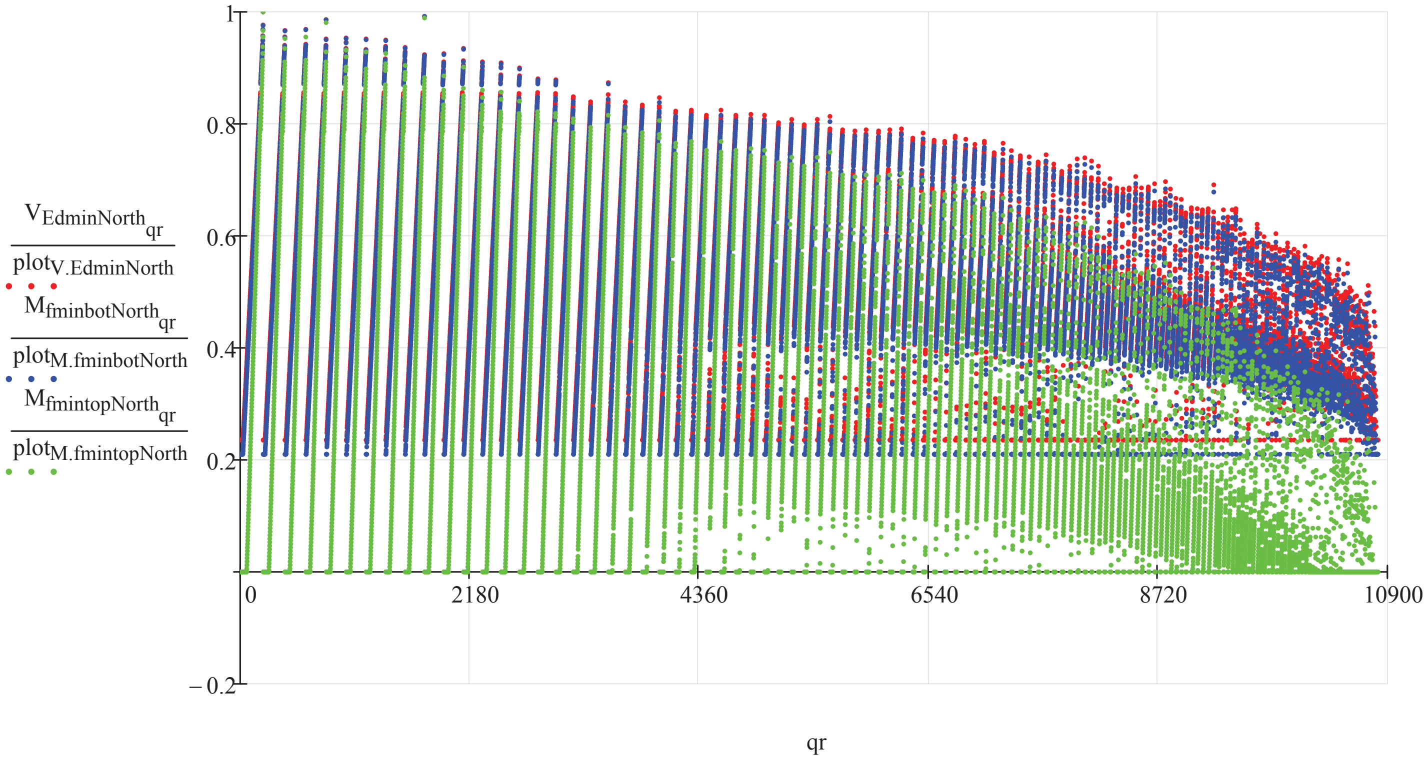
$plot_{V.EdminNorth} := \max(V_{EdminNorth})$ $plot_{M.fminbotNorth} := \max(M_{fminbotNorth})$ $plot_{M.fmintopNorth} := \max(M_{fmintopNorth})$

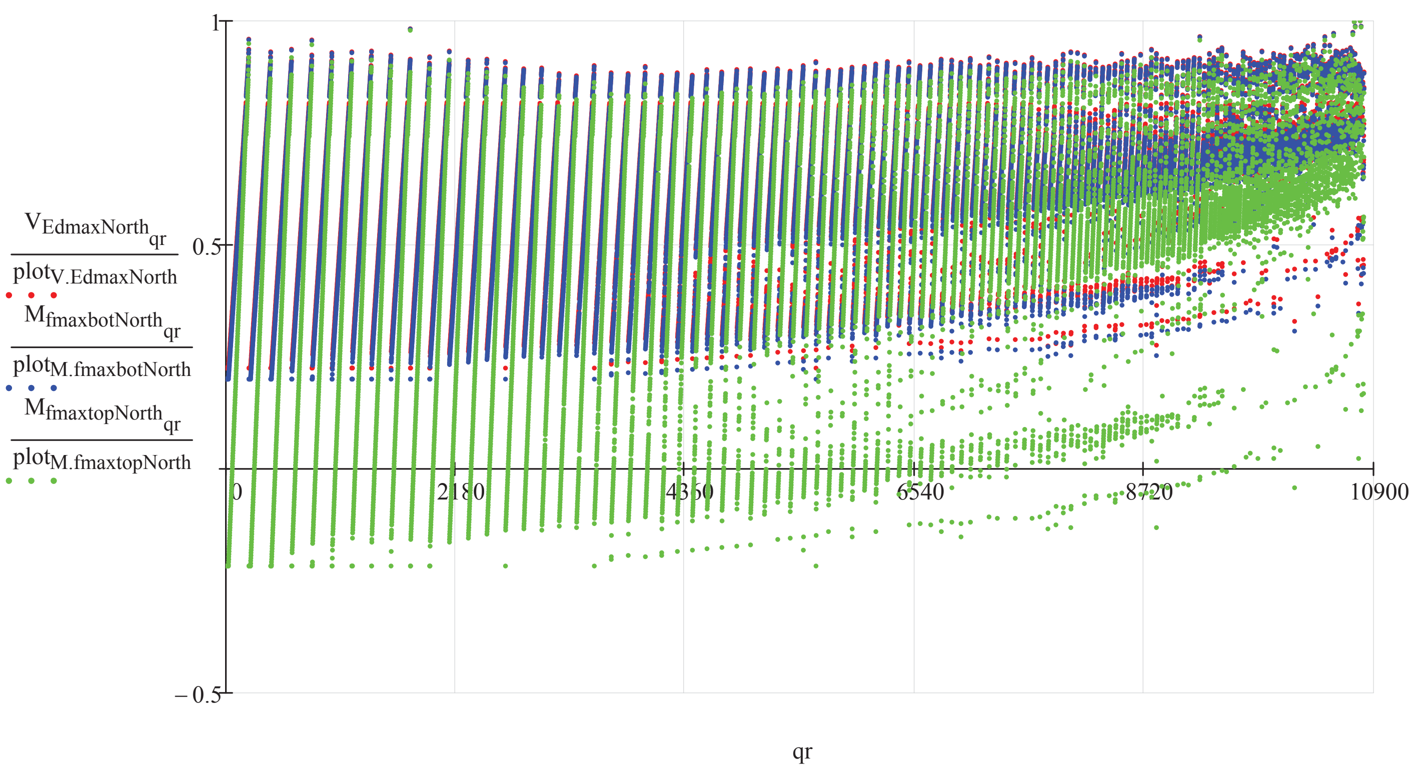
$plot_{V.EdmaxNorth} := \max(V_{EdmaxNorth})$ $plot_{M.fmaxbotNorth} := \max(M_{fmaxbotNorth})$ $plot_{M.fmaxtopNorth} := \max(M_{fmaxtopNorth})$

$plot_{V.EdminNorth} = 1006 \cdot kip$ $plot_{M.fminbotNorth} = 11735 \cdot kip \cdot ft$ $plot_{M.fmintopNorth} = 7934 \cdot kip \cdot ft$

$plot_{V.EdmaxNorth} = 1053 \cdot kip$ $plot_{M.fmaxbotNorth} = 12302 \cdot kip \cdot ft$ $plot_{M.fmaxtopNorth} = 8403 \cdot kip \cdot ft$

Results in graphical form:





I. Transformed Section Analysis at Critical Section

The neutral axis depth in the cracked section is governed by the following cubic equation:

$CUBIC := 0$

Given $\quad x_{cr} := 12.67 \cdot in$

$$CUBIC = \frac{a \cdot x_{cr}^{3}}{3 \cdot h_c} + \frac{B \cdot x_{cr}^{2}}{2} - n_{mod} \cdot A_s \cdot \left(d_{face2} - x_{cr}\right)$$

$x_{cr} := Find\left(x_{cr}\right) \quad x_{cr} = 12.67 \cdot in$

Moment of intertia of transformed section after the onset of cracking:

$$I_{CR} := \frac{B \cdot x_{cr}^{3}}{3} + \frac{2 \cdot \left(\frac{x_{cr} \cdot a}{h_c}\right) \cdot x_{cr}^{3}}{12} + n_{mod} \cdot A_s \cdot \left(d_{face2} - x_{cr}\right)^2$$

$I_{CR} = 73.2 \cdot ft^4$

J. Compute Concrete and Steel Stresses

Elastic beam theory prediction of minimum compressive stress in concrete:

$$\sigma_{cminNorth_{qr}} := \max\left(0psi, \frac{M_{fminbotNorth_{qr}} \cdot x_{cr}}{I_{CR}}\right)$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stminNorth_{qr}} := \max\left[0psi, \frac{n_{mod} \cdot M_{fminbotNorth_{qr}} \cdot \left(d_{face2} - x_{cr}\right)}{I_{CR}}\right]$$

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:

$$\sigma_{stmaxNorth_{qr}} := \frac{n_{mod} \cdot M_{fmaxbotNorth_{qr}} \cdot \left(d_{face2} - x_{cr}\right)}{I_{CR}}$$

K. Compute Twisting Moments and Steel Stresses

Map results from unique matrices back to full matrices:

$$b_{effminNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} b_{effNorth_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$x_{start\_minNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} x_{startNorth_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$f_{fminNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} f_{fNorth_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

$$l_{effminNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\begin{array}{|l} l_{effNorth_{qr_{north}}} \text{ if } M_{minnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{minnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right)$$

Minimum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{twist\_min_{qr}} := \min\left(b_{effminNorth_{qr}}, a - x_{start\_minNorth_{qr}}\right) \cdot f_{fminNorth_{qr}} \cdot \frac{\left(\frac{l_{effminNorth_{qr}} - C}{2}\right)^2}{2}$$

Maximum values for plotting:

$$plot_{b.effminNorth} := \max\left(b_{effminNorth}\right) = 54.5\,ft$$

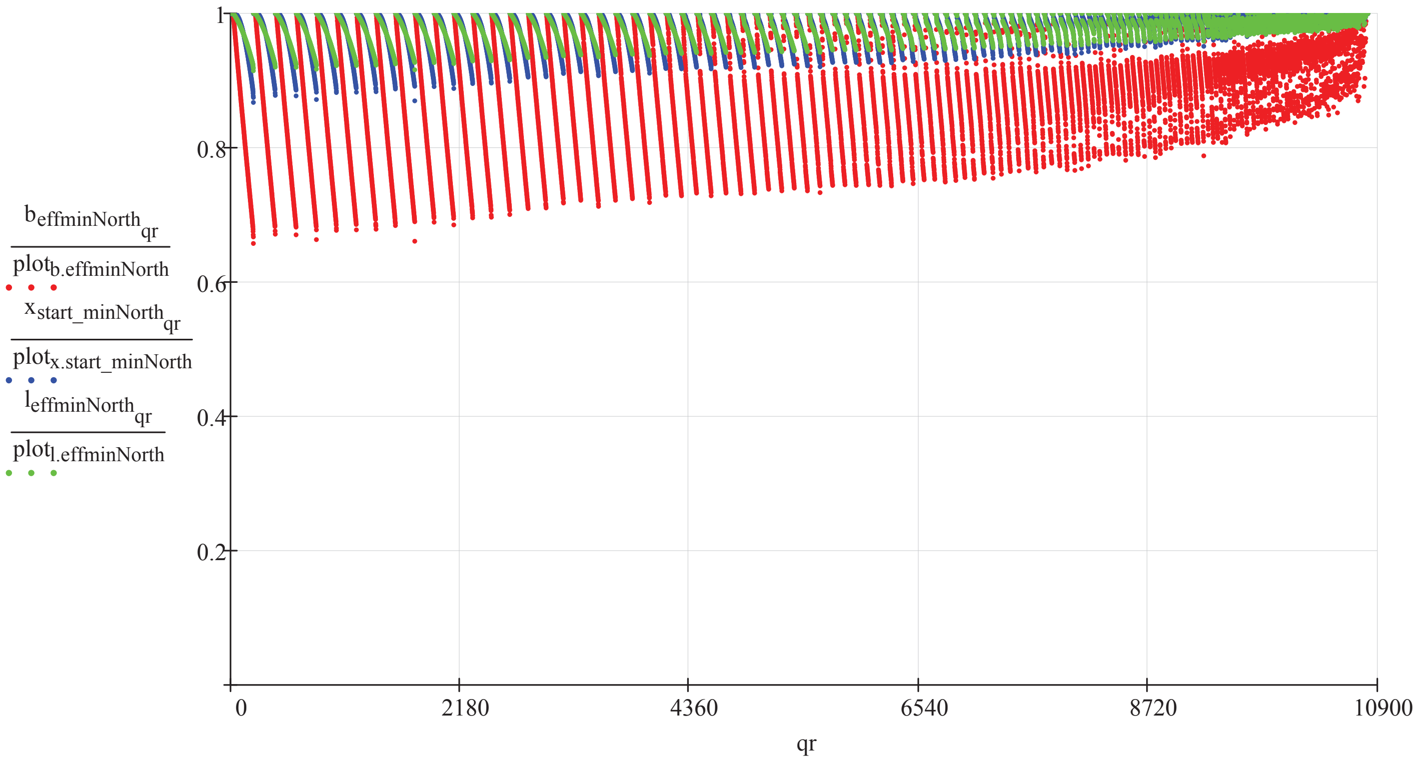
$$plot_{x.start\_minNorth} := \max\left(x_{start\_minNorth}\right) = 3.5\,ft$$

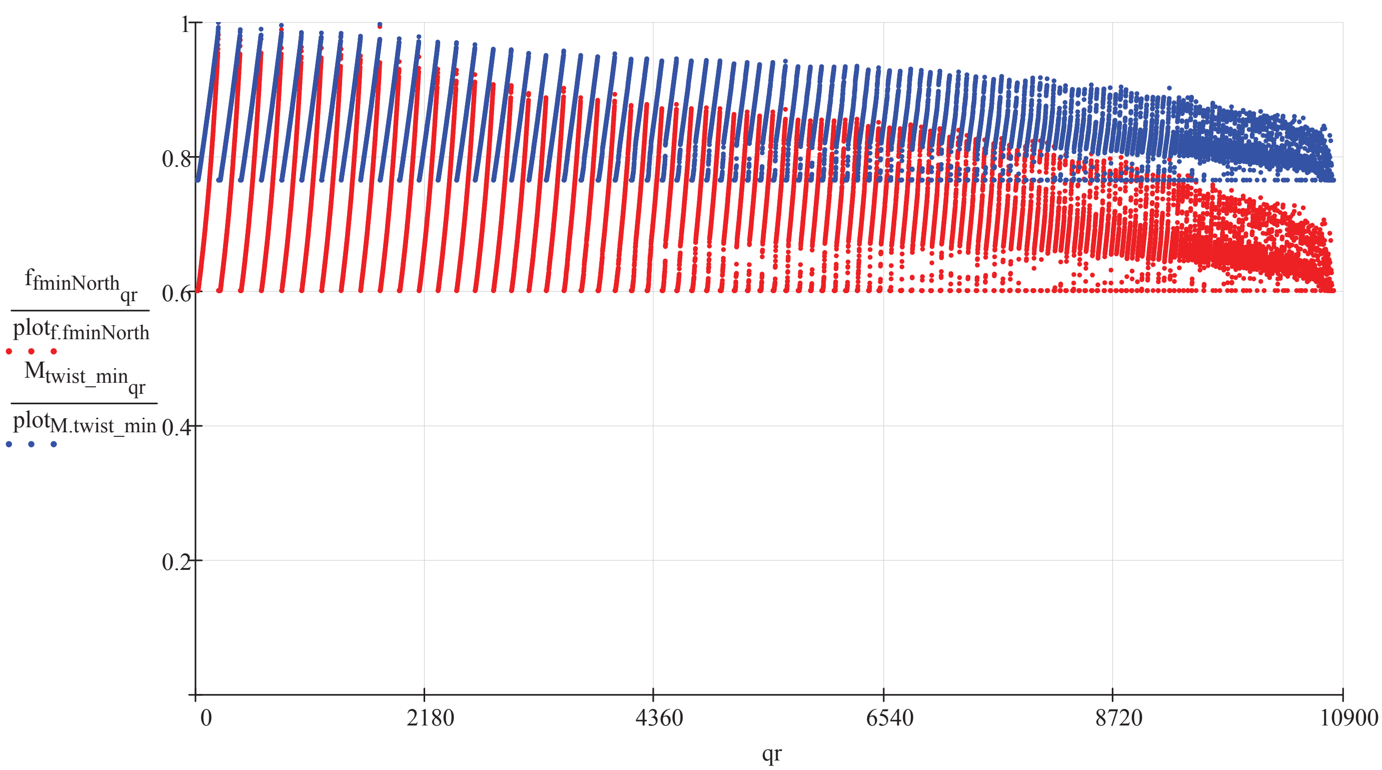
$$plot_{f.fminNorth} := \max\left(f_{fminNorth}\right) = 1769 \cdot psf$$

$$plot_{l.effminNorth} := \max\left(l_{effminNorth}\right) = 54.5\,ft$$

$$plot_{M.twist\_min} := \max\left(M_{twist\_min}\right) = 4555 \cdot kN \cdot m$$

Results in graphical form:





Map results from unique matrices back to full matrices:

$$b_{effmaxNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\left| \begin{array}{l} b_{effNorth_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right. \right)$$

$$x_{start\_maxNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\left| \begin{array}{l} x_{startNorth_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right. \right)$$

$$f_{fmaxNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\left| \begin{array}{l} f_{fNorth_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right. \right)$$

$$l_{effmaxNorth_{qr}} := \sum_{qr_{north}=0}^{qt_{north}} \left(\left| \begin{array}{l} l_{effNorth_{qr_{north}}} \text{ if } M_{maxnorth_{qr}} = MUnique_{north_{qr_{north}}} \\ 0 \text{ if } M_{maxnorth_{qr}} \neq MUnique_{north_{qr_{north}}} \end{array} \right. \right)$$

Maximum twisting moment created by transfer of bearing stresses to the pedestal width:

$$M_{twist\_max_{qr}} := \min\left(b_{effmaxNorth_{qr}}, a - x_{start\_maxNorth_{qr}}\right) \cdot f_{fmaxNorth_{qr}} \cdot \frac{\left(\frac{l_{effmaxNorth_{qr}} - C}{2}\right)^2}{2}$$

Maximum values for plotting:

$$plot_{b.effmaxNorth} := \max\left(b_{effmaxNorth}\right) = 54.5\,ft$$

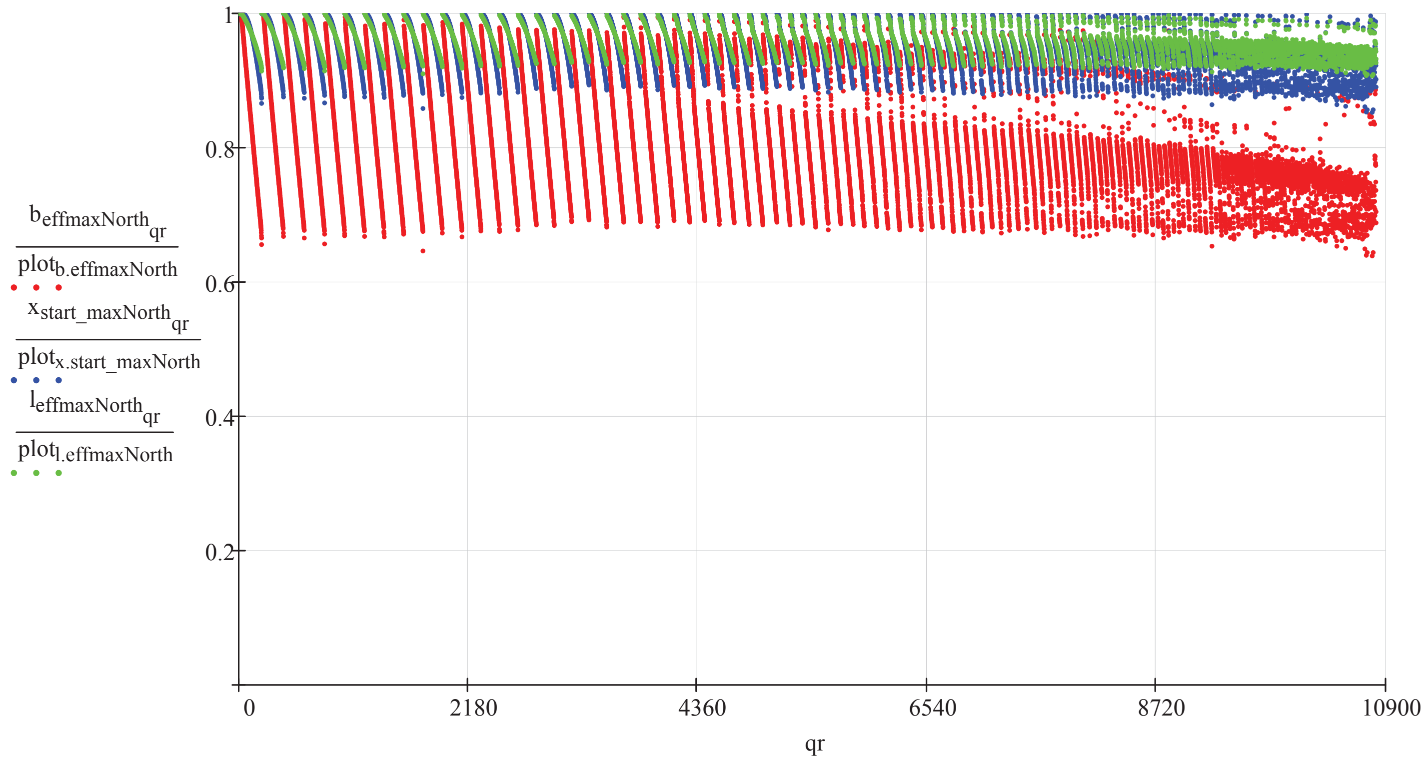
$$plot_{x.start\_maxNorth} := \max\left(x_{start\_maxNorth}\right) = 3.5\,ft$$

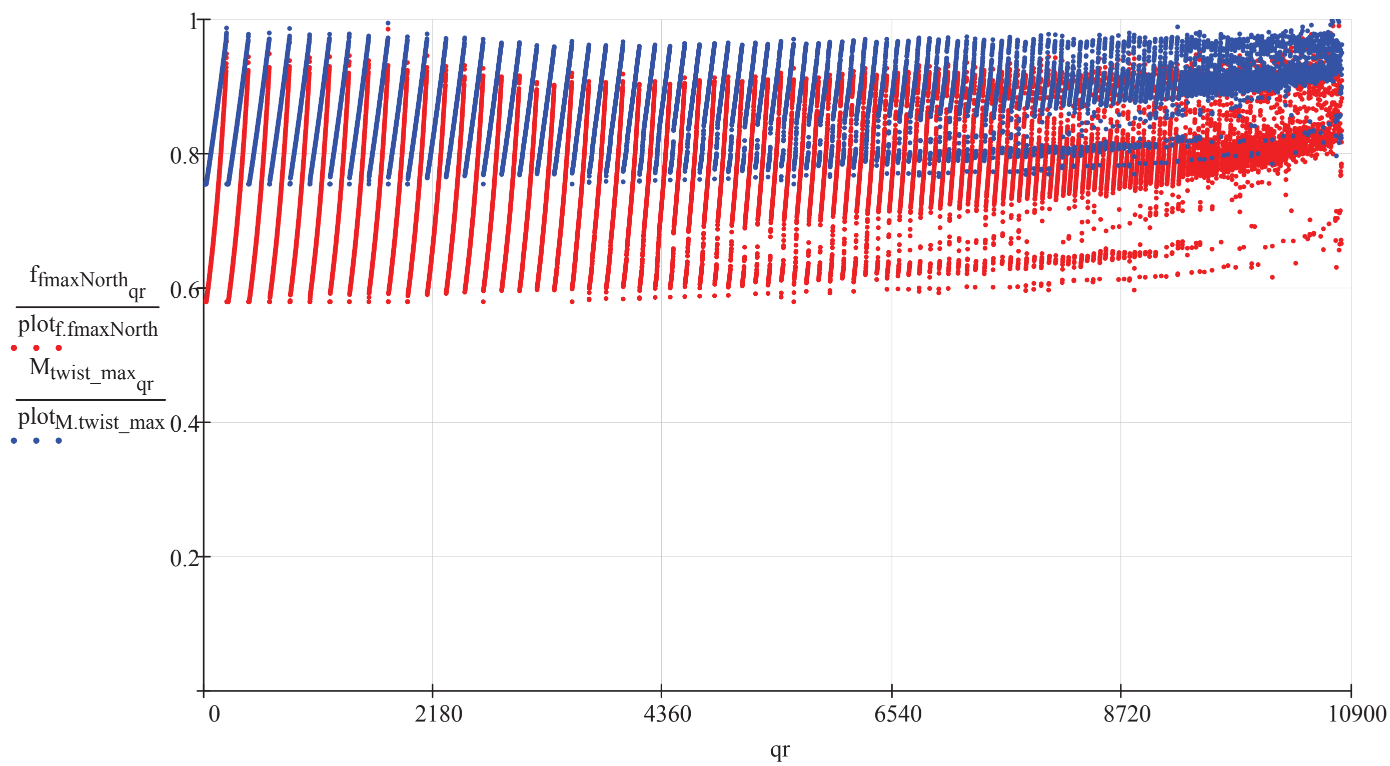
$$plot_{f.fmaxNorth} := \max\left(f_{fmaxNorth}\right) = 1836 \cdot psf$$

$$plot_{l.effmaxNorth} := \max\left(l_{effmaxNorth}\right) = 54.5\,ft$$

$$plot_{M.twist\_max} := \max\left(M_{twist\_max}\right) = 4620 \cdot kN \cdot m$$

Results in graphical form:





Area of steel available in sloped foundation section to transfer twisting moments:

$$A_{s\_twist} := \left(\frac{W_m - \frac{B}{2}}{s_{botm}}\right)\cdot\left(A_{botm}\right) + \left(\frac{\frac{D}{2} - W_m}{s_{boto}}\right)\cdot\left(A_{boto}\right) = 16.3\cdot in^2$$

Depth to reinforcement at critical section for flat portion of footing:

$$d_{face\_twist} := h_b + h_c - cc_{bot} - di_{botm} \qquad d_{face\_twist} = 61.7\cdot in$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

$$QUBIC := 0$$

Given $\quad x_{cr2} := 18.93\cdot in$

$$QUBIC = \frac{2x_{cr2}}{3}\cdot\frac{x_{cr2}}{2}\cdot\frac{x_{cr2}\cdot a}{h_c} - n_{mod}\cdot A_{s\_twist}\cdot d_{face\_twist}$$

$$x_{cr2} := \text{Find}\left(x_{cr2}\right) \; x_{cr2} = 18.93\cdot in$$

Moment of intertia of transformed section after the onset of cracking:

$$I_{CR\_twist} := \frac{\left(x_{cr2}\cdot\frac{a}{h_c}\right)\cdot x_{cr2}^{\;3}}{12} + n_{mod}\cdot A_{s\_twist}\cdot\left(d_{face\_twist} - x_{cr2}\right)^2$$

$$I_{CR\_twist} = 15.0\cdot ft^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stmin\_twist_{qr}} := \max\left[0psi, \frac{n_{mod}\cdot M_{twist\_min_{qr}}\cdot\left(d_{face\_twist} - x_{cr2}\right)}{I_{CR\_twist}}\right]$$

Elastic beam theory prediction of tensile stress in reinforcement:

$$\sigma_{stmax\_twist_{qr}} := \frac{n_{mod}\cdot M_{twist\_max_{qr}}\cdot\left(d_{face\_twist} - x_{cr2}\right)}{I_{CR\_twist}}$$

L. Concrete and Flexural Steel Stress Summary

Results in partial tabular form:

qr =	$\sigma_{cminNorth}$ = (psi)	$\sigma_{cmaxNorth}$ = (psi)	$\sigma_{stminNorth}$ = (psi)	$\sigma_{stmaxNorth}$ = (psi)	σ_{stmin_twist} = (psi)	σ_{stmax_twist} = (psi)
0	247	247	8310	8310	38154	38154
1	247	247	8310	8310	38154	38154
2	247	247	8310	8310	38154	38154
3	247	247	8310	8310	38154	38154
4	247	247	8310	8310	38154	38154
5	247	247	8310	8310	38154	38154
6	247	247	8310	8310	38154	38154
7	247	247	8310	8310	38154	38154
8	247	247	8310	8310	38154	38154
9	247	247	8310	8310	38154	38154
10	247	247	8310	8310	38154	38154
11	247	247	8310	8310	38154	38154
12	247	247	8310	8310	38154	38154
...	...	...	...	...	...	...

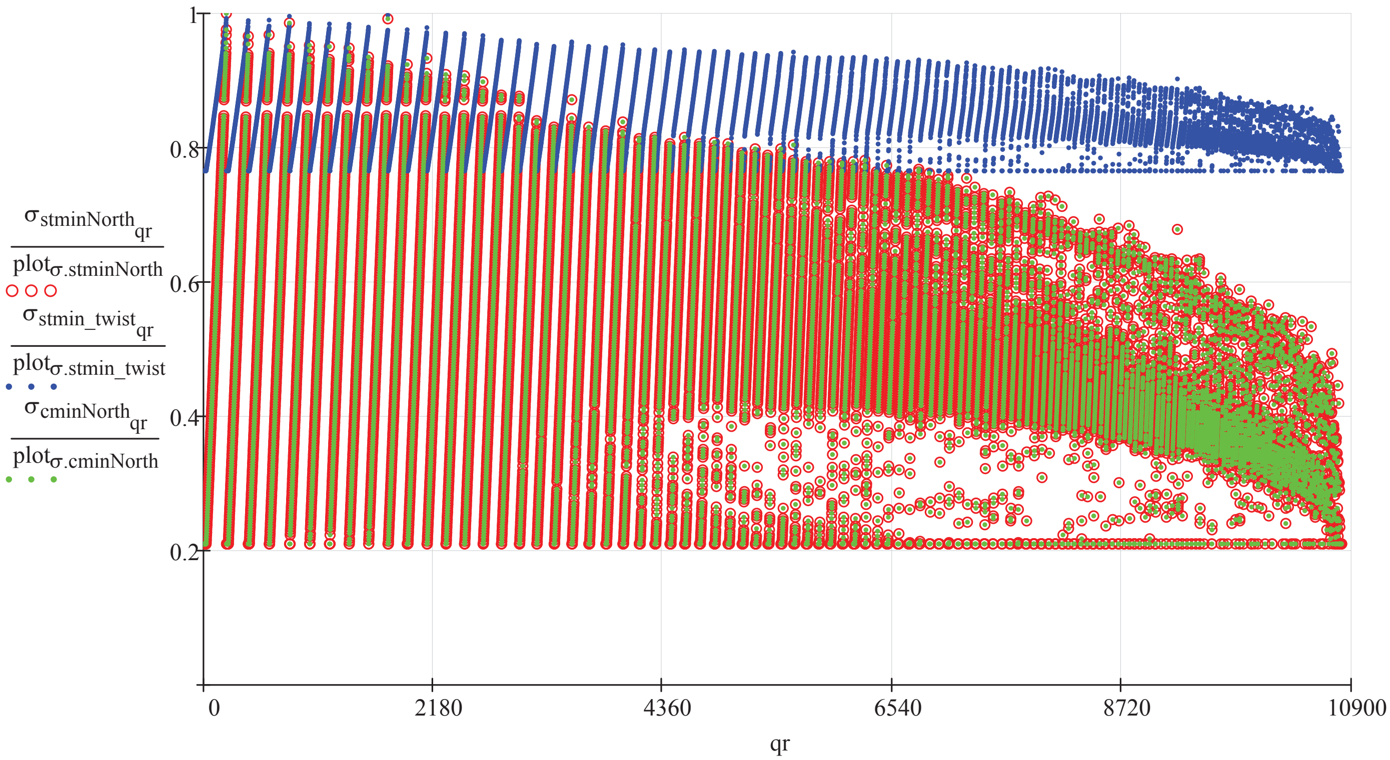
$plot_{\sigma.cminNorth} := \max\left(\sigma_{cminNorth}\right)$ $\quad plot_{\sigma.stminNorth} := \max\left(\sigma_{stminNorth}\right)$ $\quad plot_{\sigma.stmin\_twist} := \max\left(\sigma_{stmin\_twist}$

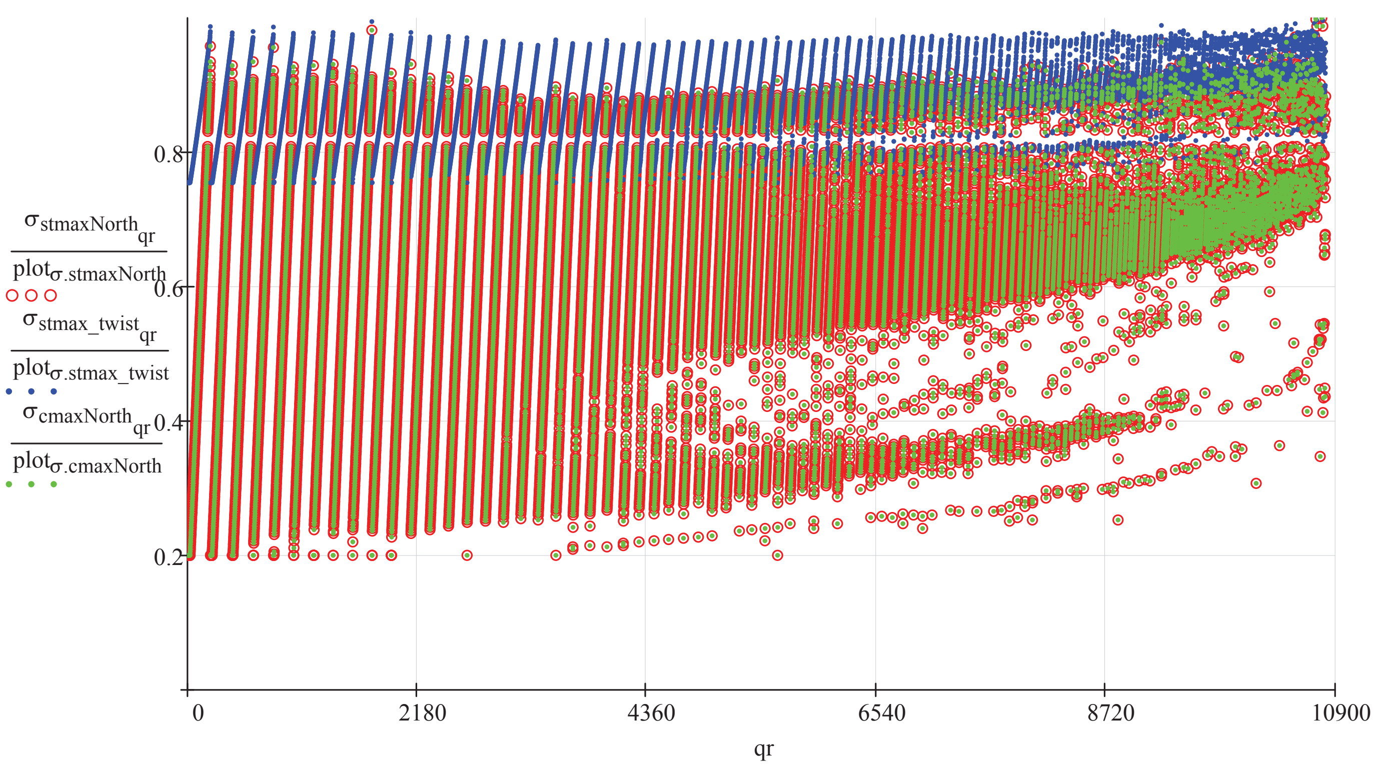
$plot_{\sigma.cmaxNorth} := \max\left(\sigma_{cmaxNorth}\right)$ $\quad plot_{\sigma.stmaxNorth} := \max\left(\sigma_{stmaxNorth}\right)$ $\quad plot_{\sigma.stmax\_twist} := \max\left(\sigma_{stmax\_twis}$

$plot_{\sigma.cminNorth} = 1176\,psi$ $\quad plot_{\sigma.stminNorth} = 39592\,psi$ $\quad plot_{\sigma.stmin\_twist} = 49841\,psi$

$plot_{\sigma.cmaxNorth} = 1233\,psi$ $\quad plot_{\sigma.stmaxNorth} = 41506\,psi$ $\quad plot_{\sigma.stmax\_twist} = 50558\,psi$

Results in graphical form:





M. Check of Fatigue in Concrete due to Compressive Stress

Material Coefficient for Reinforced Concrete: $\gamma_{cdnv} := 1.35$

Characteristic Compressive Cylinder Strength: $f_{cck} := f_c$ $\quad f_{cck} = 34.5 \cdot MPa$

Normalized Structural Compressive Strength: $f_{cn} := f_{cck} \cdot \left(1 - \frac{f_{cck}}{600 \cdot MPa}\right)$ $\quad f_{cn} = 32.5 \cdot MPa$

Design Concrete Compressive Strength: $f_{cd} := \frac{f_{cn}}{\gamma_{cdnv}}$ $\quad f_{cd} = 24.1 \cdot MPa$

Ratio between smallest and largest stresses in compression zone: $\beta := \text{if}\left(x_{cr} > 300mm, 1, 0\right)$ $\quad \beta = 1.00$

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: $f_{rd} := \alpha \cdot f_{cd}$ $\quad f_{rd} = 24.1 \cdot MPa$

Exposure Factor: $C_{1dnv} := 12$

Fatigue Strength Parameter: $C_{5c} := 1.0$

Cycles Allowed:

$$n_{callowNorth_{qr}} := 10^{C_{1dnv} \cdot \frac{\left(1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}\right)}{\left(1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}\right)}}$$

$$X_{dnvNorth_{qr}} := \frac{C_{1dnv}}{1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2dnvNorth_{qr}} := \max\left[1 + 0.2 \cdot \left(\log\left(n_{callowNorth_{qr}}\right) - X_{dnvNorth_{qr}}\right), 1.0\right]$$

$$n_{callowNorth_{qr}} := \text{if}\left[\log\left(n_{callowNorth_{qr}}\right) > X_{dnvNorth_{qr}}, 10^{C_{1dnv} \cdot C_{2dnvNorth_{qr}} \cdot \frac{\left(1 - \frac{\sigma_{cmaxNorth_{qr}}}{C_{5c} \cdot f_{rd}}\right)}{\left(1 - \frac{\sigma_{cminNorth_{qr}}}{C_{5c} \cdot f_{rd}}\right)}}, n_{callowNorth_{qr}}\right]$$

Damage: $\text{Damage}_{North_{qr}} := \frac{N_{fat_{qr}}}{n_{callowNorth_{qr}}}$

Accumulated Damage (Section 6, M108): $\text{Damage}_{totalNorth} := \sum \text{Damage}_{North}$ $\quad \boxed{\text{Damage}_{totalNorth} = 0.00}$

$\boxed{\text{if}\left(\text{Damage}_{totalNorth} < 1.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

N. Check of Fatigue in Concrete due to Shear

Concrete Area Resisting Shear: $A_{face} = 155\,ft^2$

Characteristic Tensile Strength: $f_{tk} := 0.48\cdot\left(f_{cck}\cdot MPa\right)^{0.5}$ $\qquad f_{tk} = 2.82\cdot MPa$

Normalized Tensile Strength: $f_{tn} := f_{tk}\cdot\left[1-\left(\frac{f_{tk}}{25\cdot MPa}\right)^{0.6}\right]$ $\qquad f_{tn} = 2.06\cdot MPa$

Design Tensile Strength: $f_{td} := \frac{f_{tn}}{\gamma_{cdnv}}$ $\qquad f_{td} = 1.52\cdot MPa$

Design Constants: $k_A := 100\cdot MPa$

$d_1 := 1000\cdot mm$

Anchored Reinforcement on Tensile Side: $A_s = 64\cdot in^2$

Design Factor: $k_v := \min\left(\max\left(1.5-\frac{d_{face2}}{d_1}, 1.0\right), 1.4\right)$ $\qquad k_v = 1.00$

Design Shear Strength: $V_{cd} := \min\left[0.3\cdot\left(f_{td}+\frac{k_A\cdot A_s}{\gamma_{cdnv}\cdot A_{face}}\right)\cdot A_{face}\cdot k_v, 0.6\cdot f_{td}\cdot A_{face}\cdot k_v\right]$

$V_{cd} = 1689\cdot k$

Design Shear Strength Stated in Terms of Stress: $v_{cd} := \min\left[0.3\cdot\left(f_{td}+\frac{k_A\cdot A_s}{\gamma_{cdnv}\cdot A_{face}}\right)\cdot k_v, 0.6\cdot f_{td}\cdot k_v\right]$ $\qquad v_{cd} = 76\,psi$

Cycles Allowed:

$$n_{vcallowNorth_{qr}} := 10^{\frac{C_{1dnv}\cdot\left(1-\frac{V_{EdmaxNorth_{qr}}}{C_{5c}\cdot V_{cd}}\right)}{\left(1-\frac{V_{EdminNorth_{qr}}}{C_{5c}\cdot V_{cd}}\right)}}$$

$$X_{dnvNorth_{qr}} := \frac{C_{1dnv}}{1-\frac{V_{EdminNorth_{qr}}}{C_{5c}\cdot V_{cd}}+0.1\cdot C_{1dnv}}$$

$$C_{2dnvNorth_{qr}} := \max\left[1+0.2\cdot\left(\log\left(n_{vcallowNorth_{qr}}\right)-X_{dnvNorth_{qr}}\right), 1.0\right]$$

$$n_{vcallowNorth_{qr}} := \mathrm{if}\left[\log\left(n_{vcallowNorth_{qr}}\right) > X_{dnvNorth_{qr}}, 10^{C_{1dnv} \cdot C_{2dnvNorth_{qr}} \cdot \frac{\left(1 - \frac{V_{EdmaxNorth_{qr}}}{C_{5c} \cdot V_{cd}}\right)}{\left(1 - \frac{V_{EdminNorth_{qr}}}{C_{5c} \cdot V_{cd}}\right)}}, n_{vcallowNorth_{qr}}\right]$$

Damage: $$Damage_{vNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{vcallowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108): $$Damage_{vtotalNorth} := \sum Damage_{vNorth}$$

$\boxed{Damage_{vtotalNorth} = 0.00}$

$\boxed{\mathrm{if}\left(Damage_{vtotalNorth} < 1.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

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: $f_{grtck} := f_{c28}$ $\qquad f_{grtck} = 65.5 \cdot MPa$

Normalized Structural Compressive Grout Strength: $f_{gn} := f_{grtck} \cdot \left(1 - \frac{f_{grtck}}{600 \cdot MPa}\right)$ $\qquad f_{gn} = 58.3 \cdot MPa$

Design Concrete Compressive Grout Strength: $f_{gd} := \frac{f_{gn}}{\gamma_{gdnv}}$ $\qquad f_{gd} = 43.2 \cdot MPa$

Anchor Bolt Pretension Load: $T_{pre} = 75 \cdot kip$

Flange Outside Diameter: $OD = 4556 \cdot mm$

Flange Inside Diameter: $ID = 4000 \cdot mm$

Number of Bolts: $N = 140$

Embedment plate hole diameter: $d_{hl} = 1.625 \cdot in$

Bearing area at top of grout: $A_{grt} := \frac{\pi}{4} \cdot \left(OD^2 - ID^2\right) - N \cdot \frac{\pi \cdot d_{hl}^2}{4}$ $\qquad A_{grt} = 5501 \cdot in^2$

Section modulus at top of grout:

$$S_{grt} := \frac{\pi}{32 \cdot OD}\left(OD^4 - ID^4\right) \ldots + -\left[\frac{2}{OD} \cdot \left[N \cdot \frac{\pi}{64} \cdot d_{hl}^4 + \frac{\pi}{2} \cdot d_{hl}^2 \cdot \left[\sum_{\lambda=1}^{\frac{N}{4}} \left[\left[\frac{D_i}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2 \ldots + \left[\frac{D_o}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2\right]\right]\right]\right]$$

$S_{grt} = 218440 \cdot in^3$

Minimum grout fatigue stress due to wind: $b_{fat\_grt\_minnorth_{qr}} := \frac{\left(M_{minnorth_{qr}}\right)}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$

Maximum grout fatigue stress due to wind: $b_{fat\_grt\_maxnorth_{qr}} := \frac{\left(M_{maxnorth_{qr}}\right)}{S_{grt}} + \frac{W_{mean}}{A_{grt}} + \frac{T_{pre} \cdot N}{A_{grt}}$

Cycles Allowed:

$$n_{brg\_grt\_allowNorth_{qr}} := 10^{C_{1gdnv}\cdot\frac{\left(1-\frac{b_{fat\_grt\_maxnorth_{qr}}}{C_{5g}\cdot f_{gd}}\right)}{\left(1-\frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g}\cdot f_{gd}}\right)}}$$

$$X_{brg\_grt\_dnvNorth_{qr}} := \frac{C_{1gdnv}}{1-\frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g}\cdot f_{gd}}+0.1\cdot C_{1gdnv}}$$

$$C_{2brg\_grt\_dnvNorth_{qr}} := \max\left[1+0.2\cdot\left(\log\left(n_{brg\_grt\_allowNorth_{qr}}\right)-X_{brg\_grt\_dnvNorth_{qr}}\right),1.0\right]$$

$$n_{brg\_grt\_allowNorth_{qr}} := \mathrm{if}\left[\log\left(n_{brg\_grt\_allowNorth_{qr}}\right) > X_{brg\_grt\_dnvNorth_{qr}}, 10^{C_{1gdnv}\cdot C_{2brg\_grt\_dnvNorth_{qr}}\cdot\frac{\left(1-\frac{b_{fat\_grt\_maxnorth_{qr}}}{C_{5g}\cdot f_{gd}}\right)}{\left(1-\frac{b_{fat\_grt\_minnorth_{qr}}}{C_{5g}\cdot f_{gd}}\right)}}, n_{brg\_grt\_allowNo}\right.$$

Damage:

$$Damage_{brg\_grt\_North_{qr}} := \frac{N_{fat_{qr}}}{n_{brg\_grt\_allowNorth_{qr}}}$$

Accumulated Damage
(Section 6, M108):

$$Damage_{brg\_grt\_totalNorth} := \sum Damage_{brg\_grt\_North}$$

$$\boxed{Damage_{brg\_grt\_totalNorth} = 0.06}$$

$$\boxed{\mathrm{if}\left(Damage_{brg\_grt\_totalNorth} < 1.0,\text{"OK"},\text{"NG"}\right) = \text{"OK"}}$$

P. Check of Fatigue in Pedestal Bearing Stress

Characteristic Compressive Cylinder Strength: $f_{cckp} := f_{cp}$ $f_{cckp} = 34.5 \cdot MPa$

Normalized Structural Compressive Strength: $f_{cnp} := f_{cckp} \cdot \left(1 - \frac{f_{cckp}}{600 \cdot MPa}\right)$ $f_{cnp} = 32.5 \cdot MPa$

Design Concrete Compressive Strength: $f_{cdp} := \frac{f_{cnp}}{\gamma_{cdnv}}$ $f_{cdp} = 24.1 \cdot MPa$

Grout thickness: $t_{g\_fat} := t_{gr}$ $t_{g\_fat} = 2.00 \cdot in$

Bearing area at bottom of grout: $A_1 := \frac{\pi}{4} \cdot \left[\left(OD + t_{g\_fat}\right)^2 - \left(ID - t_{g\_fat}\right)^2\right] - N \cdot \frac{\pi \cdot d_{SDR}^2}{4}$ $A_1 = 6452 \cdot in^2$

Section modulus at bottom of grout:

$$S_1 := \frac{\pi}{32 \cdot \left(OD + t_{g\_fat}\right)} \left[\left(OD + t_{g\_fat}\right)^4 - \left(ID - t_{g\_fat}\right)^4\right] \ldots$$
$$+ \left[\frac{2}{\left(OD + t_{g\_fat}\right)} \cdot \left[N \cdot \frac{\pi}{64} \cdot d_{SDR}^4 + \frac{\pi}{2} \cdot d_{SDR}^2 \cdot \left[\sum_{\lambda=1}^{\frac{N}{4}} \left[\left[\frac{D_i}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2 \ldots + \left[\frac{D_o}{2} \cdot \cos\left[\frac{2 \cdot \pi}{N} \cdot (2 \cdot \lambda - 1)\right]\right]^2\right]\right]\right]\right]$$

$S_1 = 253862 \cdot in^3$

Minimum fatigue stress due to wind: $b_{fatminnorth_{qr}} := \frac{\left(M_{minnorth_{qr}}\right)}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$

Maximum fatigue stress due to wind: $b_{fatmaxnorth_{qr}} := \frac{\left(M_{maxnorth_{qr}}\right)}{S_1} + \frac{W_{mean}}{A_1} + \frac{T_{pre} \cdot N}{A_1}$

Bearing strip (radial) at bottom of grout: $A_{strip\_1} := \frac{\left(OD + t_{g\_fat}\right) - \left(ID - t_{g\_fat}\right)}{2}$ $A_{strip\_1} = 12.9 \cdot in$

Angle of bearing within concrete: $\alpha_{DNV} := \operatorname{atan}\left(\frac{1}{2}\right)$ $\alpha_{DNV} = 26.6 \cdot deg$

Critical bearing angle within concrete that defines the pedestal bottom edge: $\alpha_{critical} := \min\left[\operatorname{atan}\left[\frac{\frac{C-(OD+t_{g\_fat})}{2}}{h_p}\right], \operatorname{atan}\left(\frac{1}{2}\right)\right]$ $\alpha_{critical} = 16.1 \cdot deg$

Bearing strip (radial) within concrete at base of pedestal: $A_{strip\_2} := \min\left(A_{strip\_1} + 2 \cdot h_p \cdot \tan\left(\min\left(\alpha_{DNV}, \alpha_{critical}\right)\right), 4 \cdot A_{strip\_1}, A_{strip\_1} + h_p\right)$

$A_{strip\_2} = 47.6 \cdot in$

Design Bearing Capacity
Stated in Terms of Stress: $F_{cd} := f_{cdp} \cdot \min\left[\left(\frac{A_{strip\_2}}{A_{strip\_1}}\right)^{\frac{1}{3}}, 1.3\right]$ $F_{cd} = 4538 \cdot psi$

Cycles Allowed:

$$n_{brgallowNorth_{qr}} := 10^{C_{1dnv} \cdot \frac{\left(1 - \frac{b_{fatmaxnorth_{qr}}}{C_{5c} \cdot F_{cd}}\right)}{\left(1 - \frac{b_{fatminnorth_{qr}}}{C_{5c} \cdot F_{cd}}\right)}}$$

$$X_{brgdnvNorth_{qr}} := \frac{C_{1dnv}}{1 - \frac{b_{fatminnorth_{qr}}}{C_{5c} \cdot F_{cd}} + 0.1 \cdot C_{1dnv}}$$

$$C_{2brgdnvNorth_{qr}} := \max\left[1 + 0.2 \cdot \left(\log\left(n_{brgallowNorth_{qr}}\right) - X_{brgdnvNorth_{qr}}\right), 1.0\right]$$

$$n_{brgallowNorth_{qr}} := \mathrm{if}\left[\log\left(n_{brgallowNorth_{qr}}\right) > X_{brgdnvNorth_{qr}}, 10^{C_{1dnv} \cdot C_{2brgdnvNorth_{qr}} \cdot \frac{\left(1 - \frac{b_{fatmaxnorth_{qr}}}{C_{5c} \cdot F_{cd}}\right)}{\left(1 - \frac{b_{fatminnorth_{qr}}}{C_{5c} \cdot F_{cd}}\right)}}, n_{brgallowNorth_{c}}\right.$$

Damage: $Damage_{brgNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{brgallowNorth_{qr}}}$

Accumulated Damage
(Section 6, M108): $Damage_{brgtotalNorth} := \sum Damage_{brgNorth}$ $\boxed{Damage_{brgtotalNorth} = 0.01}$

$\boxed{\mathrm{if}\left(Damage_{brgtotalNorth} < 1.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

Q. Check of Fatigue in Pedestal Bursting Reinforcement

Design factors: $C_3 := 19.6$

$C_4 := 6$

Characteristic strength of reinforcement: $f_{skb} := f_{yv}$ $\qquad f_{skb} = 60 \cdot ksi$

Material coefficient for reinforcement: $\gamma_s := 1.00$

Hoop bar size: $Size_{burst} := 5$

Area of radial steel bursting reinforcement located between loaded area and distribution area:

$$A_{burst} := \left(2\,\text{vlookup}\left(Size_{burst}, ACI\_bar\_table, 2\right)\right)_0 \cdot in^2 = 0.62 \cdot in^2$$

Bursting load reduction factor: $\xi := 1 - \frac{A_{strip\_1}}{A_{strip\_2}}$ $\qquad \xi = 0.73$ $\qquad$ Section 6, L110

Minimum fatigue force in bursting reinforcement due to wind:

$$F_{f\_minnorth_{qr}} := \frac{\cos\left(90deg - \min\left(\alpha_{DNV}, \alpha_{critical}\right)\right)}{2 \cdot \cos\left(\min\left(\alpha_{DNV}, \alpha_{critical}\right)\right)} \cdot \left(b_{fatminnorth_{qr}} \cdot \frac{A_1}{\frac{N}{2}}\right) \cdot \xi$$

Maximum fatigue force in bursting reinforcement due to wind:

$$F_{f\_maxnorth_{qr}} := \frac{\cos\left(90deg - \min\left(\alpha_{DNV}, \alpha_{critical}\right)\right)}{2 \cdot \cos\left(\min\left(\alpha_{DNV}, \alpha_{critical}\right)\right)} \cdot \left(b_{fatmaxnorth_{qr}} \cdot \frac{A_1}{\frac{N}{2}}\right) \cdot \xi$$

Check maximum tensile force in steel:

$$\boxed{CheckForce_{Northmax1} := \text{if}\left(\max\left(F_{f\_maxnorth}\right) > \frac{f_{skb} \cdot A_{burst}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}}$$

Tensile force range in steel:

$$\Delta F_{fNorth_{qr}} := \max\left[1 \cdot lbf, \left(F_{f\_maxnorth_{qr}} - F_{f\_minnorth_{qr}}\right)\right]$$

Cycles Allowed:

$$n_{fallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta F_{fNorth_{qr}}}{A_{burst} \cdot MPa}\right)\right)}$$

$$n_{fallowNorth_{qr}} := \text{if}\left(n_{fallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{fallowNorth_{qr}}\right)$$

Damage:

$$Damage_{burstNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{fallowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108):

$$Damage_{bursttotalNorth} := \sum Damage_{burstNorth}$$

$$\boxed{Damage_{bursttotalNorth} = 0.01}$$

$$\boxed{\text{if}\left(Damage_{bursttotalNorth} < 0.5, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$$

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: $\Delta\sigma_{stNorth_{qr}} := \max\left[1\cdot psi, \left(\sigma_{stmaxNorth_{qr}} - \sigma_{stminNorth_{qr}}\right)\right]$

Characteristic strength of reinforcement: $f_{sk} := f_y$

Material coefficient for reinforcement: $\gamma_s = 1.00$

Check maximum tensile stress in steel: $\boxed{CheckStress_{Northmax2} := if\left(\max\left(\sigma_{stmaxNorth}\right) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}}$

Cycles Allowed:

$$n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4\cdot\log\left(\frac{\Delta\sigma_{stNorth_{qr}}}{MPa}\right)\right)}$$

$$n_{sallowNorth_{qr}} := if\left(n_{sallowNorth_{qr}} > 2\cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:

$$Damage_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108):

$$Damage_{stotalNorth} := \sum Damage_{sNorth}$$

$\boxed{Damage_{stotalNorth} = 0.12}$

$\boxed{if\left(Damage_{stotalNorth} < 0.5, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

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: $\Delta\sigma_{stNorth\_normal_{qr}} := \max\left[1 \cdot psi, \left(\sigma_{stmax\_twist_{qr}} - \sigma_{stmin\_twist_{qr}}\right)\right]$

Characteristic strength of reinforcement: $f_{sk} := f_y$

Material coefficient for reinforcement: $\gamma_s = 1.00$

Check maximum tensile stress in steel: $\boxed{CheckStress_{Northmax3} := \text{if}\left(\max\left(\sigma_{stmax\_twist}\right) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}}$

Cycles Allowed:

$$n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{stNorth\_normal_{qr}}}{MPa}\right)\right)}$$

$$n_{sallowNorth_{qr}} := \text{if}\left(n_{sallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:

$$Damage_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$$

Accumulated Damage (Section 6, M108):

$$Damage_{stotalNorth\_normal} := \sum Damage_{sNorth}$$

$\boxed{Damage_{stotalNorth\_normal} = 0.00}$

$\boxed{\text{if}\left(Damage_{stotalNorth\_normal} < 0.5, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

T. Check of Fatigue in Top Steel due to Tensile Stress

Depth to reinforcement at critical section for flat portion of footing:

$$d_{face2} := h_b + h_c - cc_{top} - 1.5di_{topm} - 1in \qquad d_{face2} = 61.1 \cdot in$$

Depth to reinforcement at critical section for sloped portion of footing:

$$d_{face3} := \frac{d_{edge} + d_{face2}}{2} \qquad d_{face3} = 56.12 \cdot in$$

The neutral axis depth in the cracked section is governed by the following cubic equation:

Given $\quad x_{cr} := 12.47 \cdot in$

$$CUBIC = \frac{w_f \cdot x_{cr}^2}{2} - n_{mod} \cdot \left(\frac{B}{s_{topm}} \cdot A_{topm}\right) \cdot \left(d_{face2} - x_{cr}\right) \ldots$$
$$+ -n_{mod} \cdot \left[\frac{\left(w_f - B\right)}{s_{topm}} \cdot A_{topm}\right] \cdot \left(d_{face3} - x_{cr}\right)$$

$$x_{cr} := Find\left(x_{cr}\right) \qquad x_{cr} = 12.47 \cdot in$$

Moment of intertia of transformed section after the onset of cracking:

$$I_{CR} := \frac{w_f \cdot x_{cr}^3}{3} + n_{mod} \cdot \left(\frac{B}{s_{topm}} \cdot A_{topm}\right) \cdot \left(d_{face2} - x_{cr}\right)^2 \ldots$$
$$+ n_{mod} \cdot \left[\frac{\left(w_f - B\right)}{s_{topm}} \cdot A_{topm}\right] \cdot \left(d_{face3} - x_{cr}\right)^2$$

$$I_{CR} = 79.4 \cdot ft^4$$

Elastic beam theory prediction of minimum tensile stress in reinforcement:

$$\sigma_{stmintopNorth_{qr}} := \frac{n_{mod} \cdot M_{fmintopNorth_{qr}} \cdot \left(d_{face2} - x_{cr}\right)}{I_{CR}}$$

Elastic beam theory prediction of maximum tensile stress in reinforcement:

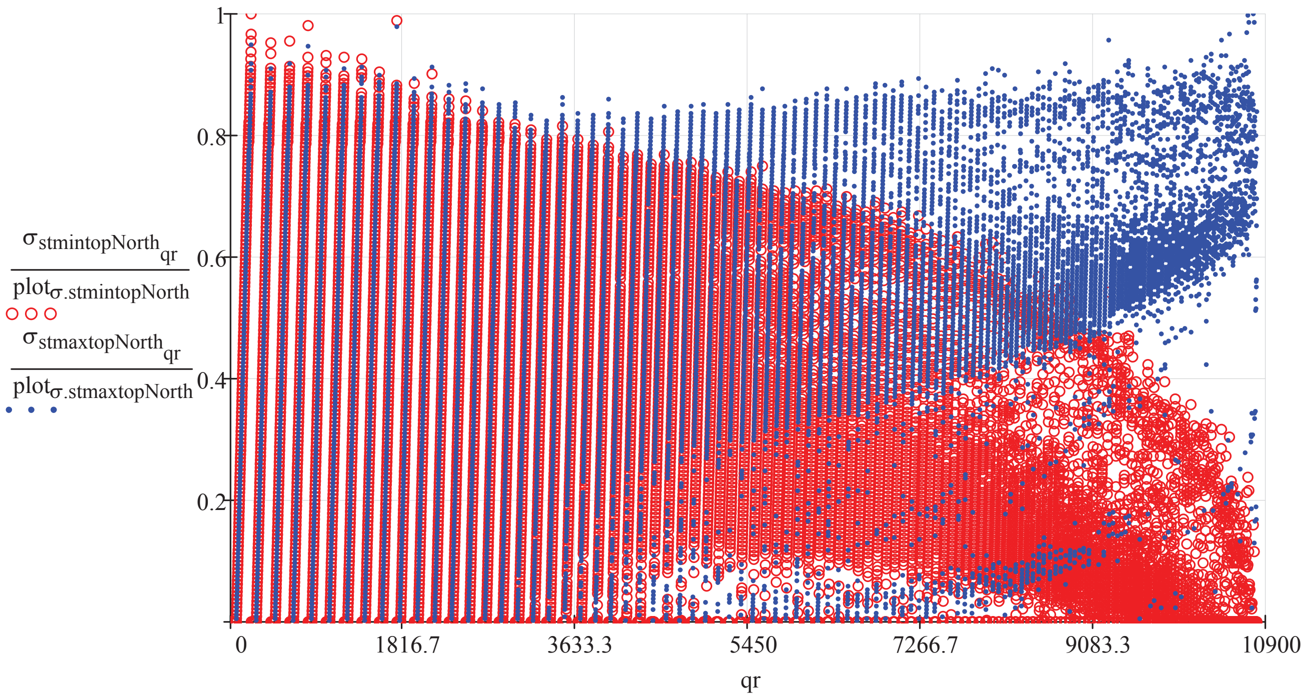
$$\sigma_{stmaxtopNorth_{qr}} := \frac{n_{mod} \cdot M_{fmaxtopNorth_{qr}} \cdot \left(d_{face2} - x_{cr}\right)}{I_{CR}}$$

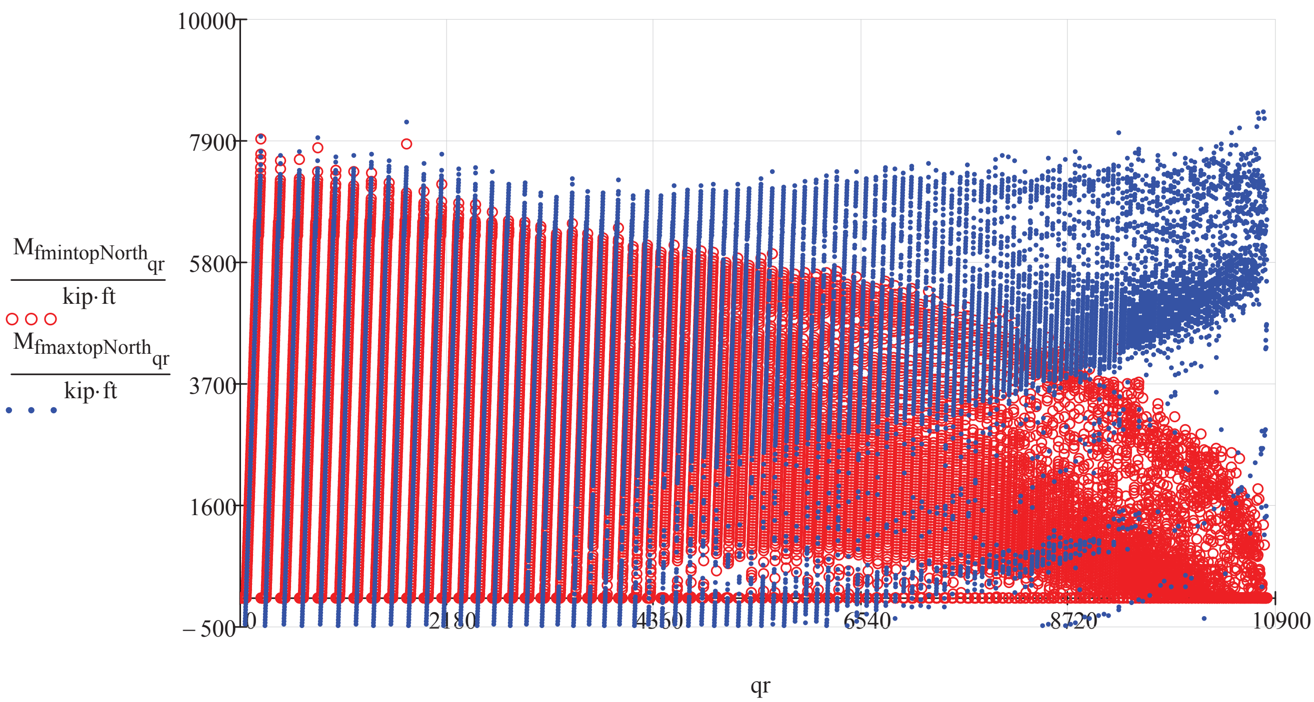
$plot_{\sigma.stmintopNorth} := \max(\sigma_{stmintopNorth})$

$plot_{\sigma.stmaxtopNorth} := \max(\sigma_{stmaxtopNorth})$

$plot_{\sigma.stmintopNorth} = 25294\,psi$

$plot_{\sigma.stmaxtopNorth} = 26789\,psi$





Design factors: $C_3 = 19.60$

$C_4 = 6.00$

Tensile stress range in steel: $\Delta\sigma_{sttopNorth_{qr}} := \max\left[1 \cdot psi, \left(\sigma_{stmaxtopNorth_{qr}} - \sigma_{stmintopNorth_{qr}}\right)\right]$

Characteristic strength of reinforcement: $f_{sk} := f_y$

Material coefficient for reinforcement: $\gamma_s = 1.00$

Check maximum tensile stress in steel: $\boxed{CheckStress_{Northmax4} := if\left(\max\left(\sigma_{stmaxtopNorth}\right) > \frac{f_{sk}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}}$

Cycles Allowed:

$$n_{sallowNorth_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\Delta\sigma_{sttopNorth_{qr}}}{MPa}\right)\right)}$$

$$n_{sallowNorth_{qr}} := if\left(n_{sallowNorth_{qr}} > 2 \cdot 10^8, 10^{307}, n_{sallowNorth_{qr}}\right)$$

Damage:

$$Damage_{sNorth_{qr}} := \frac{N_{fat_{qr}}}{n_{sallowNorth_{qr}}}$$

Accumulated Damage (Section 6, M107):

$$Damage_{stoptotalNorth} := \sum Damage_{sNorth}$$

$\boxed{Damage_{stoptotalNorth} = 0.10}$

$\boxed{if\left(Damage_{stoptotalNorth} < 0.5, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$

U. Check Fatigue Tensile Strength of Double Verts

Characteristic strength of reinforcement: $f_{skv} := f_{yv}$ $\quad f_{skv} = 60 \cdot ksi$

Material coefficient for reinforcement: $\gamma_s = 1.00$

Area of reinforcement: $A_r + A_{rH} = 1.79 \cdot in^2$

Number of bars: $n_z = 140$

Inner bolt circle diameter: $D_i = 4130 \cdot mm$

Outer bolt circle diameter: $D_o = 4426 \cdot mm$

Average bolt circle diameter: $D_{avg} = 4278 \cdot mm$

Minimum fatigue force due to wind:

$$P_{fat\_min\_S_{qr}} := \max\left[0psi, \left[\frac{4 \cdot \left(M_{minnorth_{qr}}\right)}{n_z \cdot D_{avg}} - \frac{W_{mean} + W_p}{n_z}\right]\right]$$

Maximum fatigue force due to wind:

$$P_{fat\_max\_S_{qr}} := \left[\frac{4 \cdot \left(M_{maxnorth_{qr}}\right)}{n_z \cdot D_{avg}} - \frac{W_{mean} + W_p}{n_z}\right]$$

Maximum stress in reinforcement:

$$\sigma_{Zmax_{qr}} := \frac{P_{fat\_max\_S_{qr}}}{A_r + A_{rH}}$$

Check maximum tensile stress in steel:

$$\boxed{CheckStress_{Northmax5} := \text{if}\left(\max\left(\sigma_{Zmax}\right) > \frac{f_{skv}}{\gamma_s}, \text{"No Good"}, \text{"Okay"}\right) = \text{"Okay"}}$$

Stress range in reinforcement:

$$\Delta\sigma_{Z_{qr}} := \sigma_{Zmax_{qr}} - \frac{P_{fat\_min\_S_{qr}}}{A_r + A_{rH}}$$

Cycles Allowed:

$$n_{Zallow_{qr}} := 10^{\left(C_3 - C_4 \cdot \log\left(\frac{\max\left(10^{-30} psi, \Delta\sigma_{Z_{qr}}\right)}{MPa}\right)\right)}$$

$$n_{Zallow_{qr}} := \text{if}\left(n_{Zallow_{qr}} > 2 \cdot 10^8, 10^{307}, n_{Zallow_{qr}}\right)$$

Damage:

$$Damage_{Z_{qr}} := \frac{N_{fat_{qr}}}{n_{Zallow_{qr}}}$$

Accumulated Damage
(Section 6, M108):

$$Damage_{ZTotal} := \sum Damage_Z$$

$$\boxed{Damage_{ZTotal} = 0.25}$$

$$\boxed{\text{if}\left(Damage_{ZTotal} < 0.5, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}}$$

XI. Reinforcement Steel Weight and Concrete Volume Estimate

A. Top and Bottom Reinforcement

Number of bottom bars within middle section: $n_{botm_{n+5}} = 27.00$

Number of bars across bottom of footing at section: $n_{bot_{n+5}} = 43.00$

Length of center bar: $l_{cl} := \dfrac{D - 2 \cdot cc_{top}}{2}$ $\quad l_{cl} = 30.58\,ft$

Distance from middle to end of bar for continuous bars:

$$l_{bot3_{ib}} := if\left[z_{bot_{ib}} > \frac{B}{2}, max\left[\frac{D}{2} - \left(z_{bot_{ib}} - \frac{B}{2}\right) - \sqrt{2} \cdot cc_{top}, 0\right], max\left(\frac{D}{2} - cc_{top}, 0\right)\right]$$

Distance from middle to end of bar of bar for cut off bars:

$$l_{bot4_{ib}} := max\left(\frac{D}{2} - cd_{bot}, 0\right)$$

Selection of appropriate bar end distance for bar in question:

$$l_{bot5_{ib}} := if\left(ib \cdot s_{botm} - \frac{B}{2} < cd_{bot} \wedge \frac{ib}{2} \neq trunc\left(\frac{ib}{2}\right), l_{bot4_{ib}}, l_{bot3_{ib}}\right)$$

Weight of bottom steel:

$$WT_{bot} := 2 \cdot \left[2W_{botm} \cdot l_{cl} + 4 \cdot \sum_{mm=1}^{n_{botm_{n+5}}} \left(W_{botm} \cdot l_{bot5_{mm}}\right) + 4 \cdot \sum_{nn=\left(n_{botm_{n+5}}\right)+1}^{n_{bot_{n+5}}} \left(W_{boto} \cdot l_{bot5_{nn}}\right)\right]$$

$WT_{bot} = 13.6 \cdot tonf$ $\quad WT_{bot} = 27.10 \cdot kip$

Number of top bars within middle section: $n_{topm_{n+5}} = 29.00$

Number of bars across top of footing at section: $n_{top_{n+5}} = 56.00$

Distance from middle to end of bar for continuous bars:

$$l_{top3_{it}} := \begin{cases} max\left[\dfrac{B}{2} + \sqrt{\left[h_c - \dfrac{h_c}{a} \cdot \left(z_{top_{it}} - \dfrac{B}{2}\right)\right]^2 + \left[a - \left(z_{top_{it}} - \dfrac{B}{2}\right) - \sqrt{2} \cdot cc_{top}\right]^2}, 0\right] & \text{if } z_{top_{it}} > \dfrac{B}{2} \\ max\left[\dfrac{B}{2} + \sqrt{\left(a - cc_{top}\right)^2 + h_c^{\,2}}, 0\right] & \text{otherwise} \end{cases}$$

Distance from middle to end of bar of bar for cut off bars:

$$l_{top4_{it}} := \begin{array}{|l} \text{if } z_{top_{it}} > \frac{B}{2} \\ \quad \begin{array}{|l} \max\left[\frac{B}{2} + \sqrt{\left[h_c - \frac{h_c}{a}\cdot\left(z_{top_{it}} - \frac{B}{2}\right)\right]^2 + \left[a - \left(z_{top_{it}} - \frac{B}{2}\right) - \sqrt{2}\cdot cc_{top}\right]^2}, 0\right] \text{ if } z_{top_{it}} > \frac{B}{2} + cd_{top} \\ \max\left[\frac{B}{2} + \sqrt{\left[\left[h_c - \frac{h_c}{a}\cdot\left(z_{top_{it}} - \frac{B}{2}\right)\right]\cdot\frac{a - cd_{top}}{a - \left(z_{top_{it}} - \frac{B}{2}\right) - \sqrt{2}\cdot cc_{top}}\right]^2 + \left(a - cd_{top}\right)^2}, 0\right] \text{ otherwise} \end{array} \\ \max\left[\frac{B}{2} + \sqrt{\left(a - cd_{top}\right)^2 + \left[h_c\cdot\frac{\left(a - cd_{top}\right)}{a}\right]^2}, 0\right] \text{ otherwise} \end{array}$$

Selection of appropriate bar end distance for bar in question:

$$l_{top5_{it}} := \text{if}\left(it\cdot s_{topm} - \frac{B}{2} < cd_{top} \wedge \frac{it}{2} \neq \text{trunc}\left(\frac{it}{2}\right), l_{top4_{it}}, l_{top3_{it}}\right)$$

Weight of top steel:

$$WT_{top} := 2\cdot\left[2W_{topm}\cdot l_{cl} + 4\cdot\sum_{mm=1}^{n_{topm_{n+5}}}\left(W_{topm}\cdot l_{top5_{mm}}\right) + 4\cdot\sum_{nn=\left(n_{topm_{n+5}}\right)+1}^{n_{top_{n+5}}}\left(W_{topo}\cdot l_{top5_{nn}}\right)\right]$$

$WT_{top} = 15.5\cdot tonf$ $\qquad WT_{top} = 31.08\cdot kip$

B. Pedestal Reinforcement

Pedestal rebar size: $Size_{ped} := 6$

Pedestal rebar spacing: $s_{ped} := 6 \cdot in$

Pedestal rebar diameter: $d_{ped} := vlookup(Size_{ped}, ACI\_bar\_table, 1)_0 \cdot in = 0.750 \cdot in$

Weight per foot of pedestal rebar: $W_{ped} := vlookup(Size_{ped}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft = 1.502 \cdot \frac{lbf}{ft}$

Number of bars across top of pedestal on one half side: $n_{ped} := trunc\left(\frac{0.5C - cc_{top}}{s_{ped}}\right)$ $\quad n_{ped} = 17.00$

Pedestal bar counter: $ip := 0, 1 .. n_{ped}$

Radius of rebar circle: $r := \frac{C}{2} - cc_{top}$ $\quad r = 8.83\,ft$

Cumulative spacing from centerline to indiviudal bars: $y_{ip} := ip \cdot s_{ped}$

Length of individual bars: $l_{ped_{ip}} := 2r \cdot cos\left(asin\left(\frac{y_{ip}}{r}\right)\right)$

Total Weight of Pedestal Mat: $WT_{ped} := 2 \cdot \left[l_{ped_0} \cdot W_{ped} + 2 \cdot \left[\sum_{ip=1}^{n_{ped}} \left(l_{ped_{ip}} \cdot W_{ped} \right) \right] \right]$ $\quad WT_{ped} = 0.74 \cdot tonf$

C. Hoop Reinforcement

Hoop bar size: $Size_{hoop} := 6$

Number of Hoops: $n_{hoop} := -1 + ceil\left(\frac{h_p}{6 \cdot in}\right) = 9$

Hoop Bar Diameter: $d_{hoop} := vlookup(Size_{hoop}, ACI\_bar\_table, 1)_0 \cdot in = 0.750 \cdot in$

Weight per foot of hoop bars: $W_{hoop} := vlookup(Size_{hoop}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft = 1.502 \cdot \frac{lbf}{ft}$

Bar length: $l_{hoop} := (C - 2 \cdot cc_{top} - d_{hoop}) \cdot \pi$ $\quad l_{hoop} = 55.31\,ft$

Total Weight of Hoop Reinforcement: $WT_{hoop} := n_{hoop} \cdot l_{hoop} \cdot W_{hoop}$ $\quad WT_{hoop} = 0.37 \cdot tonf$

D. Hat Bar Reinforcement

Bar size:	$Size_H = 9$	$s_{rH} = 24.00 \cdot in$
Number of hat bars:	$n_{hat} := 0.5 \cdot n_z$	$n_{hat} = 70.00$
Weight per foot of hat bars:	$W_{hat} := vlookup\left(Size_H, ACI\_bar\_table, 3\right)_0 \cdot lbf \div ft = 3.40 \cdot \frac{lbf}{ft}$	
		$l_{hH} = 19.00 \cdot in$
Total length of hat bars:	$l_{hatbar} := 2 \cdot \left(l_{hH}\right) + 2 \cdot \left(h_e + 5in + l_{hH}\right) + s_{rH}$	$l_{hatbar} = 13.67\,ft$
Total Weight of hat bar Reinforcement:	$WT_{hatbar} := W_{hat} \cdot n_{hat} \cdot l_{hatbar}$	$WT_{hatbar} = 1.63 \cdot tonf$

E. Pedestal C-bar Reinforcement

C-bar size:	$Size_{cbars} := 6$	
Number of c-bars:	$n_{cbars} := 0.5 \cdot N$	
Weight per foot of c-bars:	$W_{cbars} := vlookup\left(Size_{cbars}, ACI\_bar\_table, 3\right)_0 \cdot lbf \div ft = 1.502 \cdot \frac{lbf}{ft}$	
Total length of c-bars:	$l_{cbars} := 2(12 \cdot in) + \left(h_p + 12in\right)$	$l_{cbars} = 8.00\,ft$
Total Weight of c-bar Reinforcement:	$WT_{cbar} := W_{cbars} \cdot n_{cbars} \cdot l_{cbars}$	$WT_{cbar} = 0.42 \cdot tonf$

F. Vertical Reinforcement

Bar size:	$Size_{vert} = 8$	
Hook length:	$l_{vert2} := l_h = 16.00 \cdot in$	$l_h = 16.00 \cdot in$
Vertical Bar Diameter:	$d_{vert} := d_r$	$d_{vert} = 1.000 \cdot in$
Number of Vertical Bars:	$n_{vert} := n_z$	$n_{vert} = 140$
Weight per foot of vertical bars:	$W_{vert} := vlookup(Size_{vert}, ACI\_bar\_table, 3)_0 \cdot lbf \div ft = 2.67 \cdot \frac{lbf}{ft}$	
Straight bar length:	$l_{vert1} := h_s + h_{pe} - cc_{bot} - cc_{top} - 2 \cdot d_{ped} \ldots + (-2 \cdot di_{botm}) - d_{vert}$	$l_{vert1} = 9.66\,ft$
Total length of vertical rebar:	$l_{vert} := l_{vert1} + 2 l_{vert2}$	$l_{vert} = 12.33\,ft$
Total Weight of Vertical Reinforcement:	$WT_{vert} := n_{vert} \cdot l_{vert} \cdot W_{vert}$	$WT_{vert} = 2.30 \cdot tonf$

G. Bursting Reinforcement

Bursting bar size:	$Size_{burst} = 5$	
Number of burst bars:	$n_{burst} := 0.5 \cdot n_z$	$n_{burst} = 70.00$
Total length of burst bars:	$l_{burstbar} := 3 \cdot (10 \cdot in) + 2 \cdot 30 \cdot in$	$l_{burstbar} = 7.5\,ft$
Weight per foot of burst bars:	$W_{burst} := vlookup(Size_{burst}, ACI\_bar\_table, 3)_0 \cdot lbf \div$	$W_{burst} = 1.043 \cdot \frac{lbf}{ft}$
Total Weight of burst bar Reinforcement:	$WT_{burst} := W_{burst} \cdot n_{burst} \cdot l_{burstbar}$	$WT_{burst} = 0.27 \cdot tonf$

H. Total Steel Weight

$f_y = 75.00 \cdot ksi$

Total weight of steel in top and bottom:	$W_{totTandB} := WT_{bot} + WT_{top}$	$\boxed{W_{totTandB} = 29.1 \cdot tonf}$
Total weight of steel:	$W_{tot} := WT_{bot} + WT_{top} + WT_{ped} + WT_{hoop} \ldots + WT_{hatbar} + WT_{cbar} + WT_{vert} + WT_{burst}$	$\boxed{W_{tot} = 34.8 \cdot tonf}$

$$\boxed{Grade60 := (W_{tot}) - W_{totTandB} = 5.7 \cdot tonf}$$

I. Total Concrete Volume

Total volume of concrete (assuming no embedments): $v_c = 460 \cdot yd^3$

Unit weight of steel: $\gamma_{stl} := 490pcf$

Total volume of steel reinforcement: $V_{tot\_rebar} := \frac{W_{tot}}{\gamma_{stl}}$ $\quad V_{tot\_rebar} = 5.3 \cdot yd^3$

Total volume of steel embedment ring: $V_{embed\_ring} := t \cdot \left[\frac{\pi}{4} \cdot \left(OD^2 - ID^2\right)\right]$ $\quad V_{embed\_ring} = 0.12 \cdot yd^3$

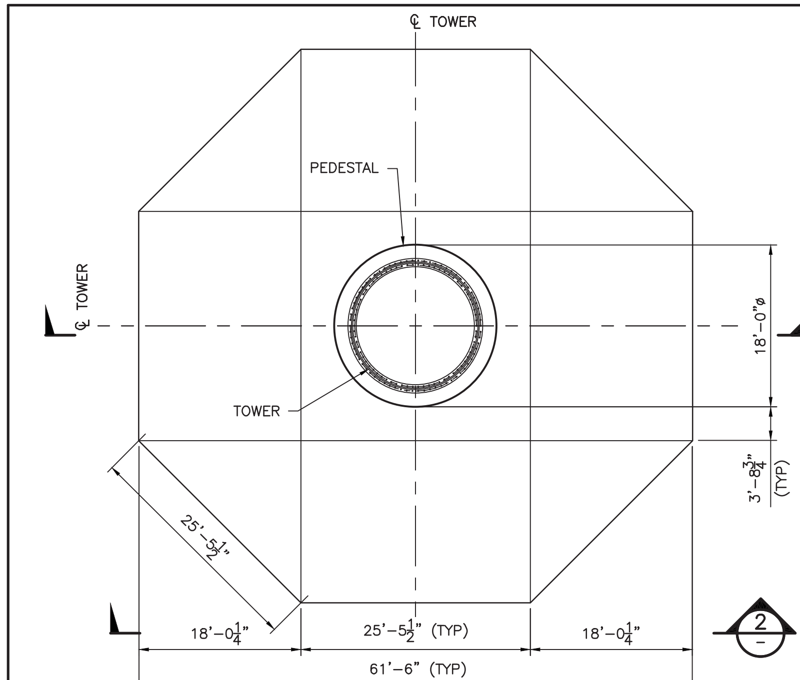
Total volume of anchor bolts: $V_{anchor\_bolts} := \left(h_b + h_c + h_p - h_e - t\right) \cdot N \cdot \left(\frac{\pi}{4} \cdot d_{SDR}^{\ 2}\right)$ $\quad V_{anchor\_bolts} = 0.83 \cdot yd^3$

Total volume of concrete (accounting for embedments): $v_{c\_redux} := v_c - V_{tot\_rebar} \cdots + -V_{embed\_ring} - V_{anchor\_bolts}$ $\quad \boxed{v_{c\_redux} = 454 \cdot yd^3}$

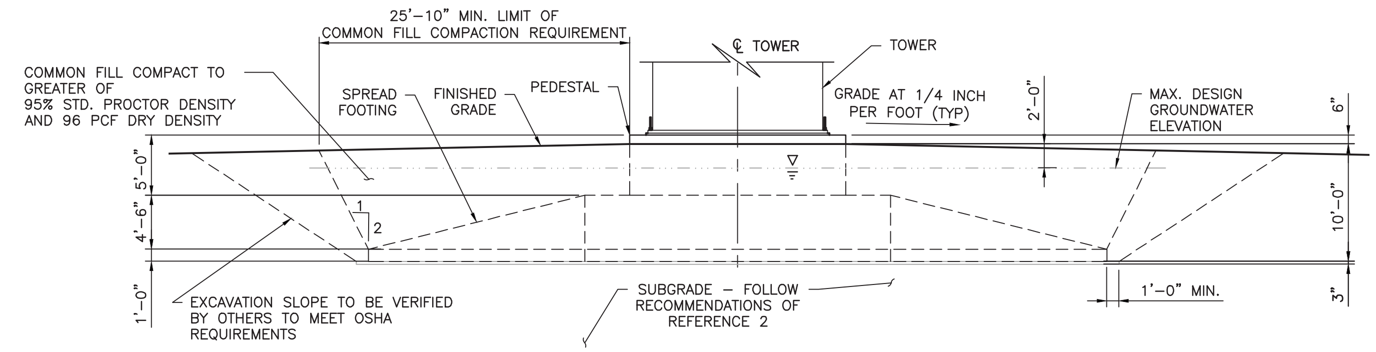
J. Summary: Shear, Reinforcing, and Fatigue

Aggregate Size:	$d_a = 0.75 \cdot in$			
	Value at Critical Section		*Maximum Value of all "Bins"*	
ACI Shear Ratio at Critical Section:	$Output_{ShearCriticalSection1} = 0.38$	< 1.0	$Check_{Shear1} = 0.39$	< 1.0
CI Shear Ratio at Critical Section:	$Output_{ShearCriticalSection2} = 0.68$	< 1.0	$Check_{Shear2} = 0.68$	< 1.0
Middle Strip Moment Ratios:	$Output_{MiddleStripRatioW} = 0.96$	$Output_{MiddleStripRatioEQ} = 1.47$	> 0.60	
Top Steel Orthogonal Ratio:	$Output_{TopSteel} = 0.78$	< 1.0		
Top Steel Cutoff Ratio:	$Output_{TopSteelCutoff} = 0.94$	< 1.0		
Top Steel 45 Degree Ratio:	$Output_{TopSteel45} = 0.81$	< 1.0		
Bottom Steel Orthogonal Ratio:	$Output_{BottomSteel} = 0.96$	< 1.0		
Bottom Steel Cutoff Ratio:	$Output_{BottomSteelCutoff} = 0.85$	< 1.0		
Bottom Steel 45 Degree Ratio:	$Output_{BottomSteel45} = 0.95$	< 1.0		
Two Way Shear Ratio:	$Output_{TwoWayShear} = 0.42$	< 1.0		
	$\gamma_{vEQ} = 0.40$	$\gamma_{vW} = 0.40$		
Concrete Compression Fatigue:	$Damage_{totalNorth} = 0.00$	< 1.0		
Concrete Shear Fatigue:	$Damage_{vtotalNorth} = 0.00$	< 1.0		
Grout Bearing Fatigue:	$Damage_{brg\_grt\_totalNorth} = 0.06$	< 1.0		
Concrete Bearing Fatigue:	$Damage_{brgtotalNorth} = 0.01$	< 1.0		$\alpha_{DNV} = 26.6 \cdot deg$
Bursting Steel Fatigue:	$Damage_{bursttotalNorth} = 0.01$	< 0.5	$A_{burst} = 0.62 \cdot in^2$	$CheckForce_{Northmax1} = "Okay"$
Steel Tension Fatigue (primary):	$Damage_{stotalNorth} = 0.12$	< 0.5	$CheckStress_{Northmax2} = "Okay"$	
Steel Tension Fatigue (normal):	$Damage_{stotalNorth\_normal} = 0.00$	< 0.5	$CheckStress_{Northmax3} = "Okay"$	
Steel Tension Fatigue (top):	$Damage_{stoptotalNorth} = 0.10$	< 0.5	$CheckStress_{Northmax4} = "Okay"$	
Steel Verts Fatigue:	$Damage_{ZTotal} = 0.25$	< 0.5	$CheckStress_{Northmax5} = "Okay"$	
Total T&B Reinforcement Weight:	$W_{totTandB} = 29.1 \cdot tonf$			
Total Reinforcement Weight:	$W_{tot} = 34.8 \cdot tonf$	$W_{tot} - W_{totTandB} = 5.74 \cdot tonf$		
Total Concrete Volume:	$v_{c\_redux} = 454 \cdot yd^3$			
Number of Bottom Bars in Middle Section:		$N_{BotMidBars} = 55$		
Number of Bottom Bars in Outer Section:		$N_{BotOutBars} = 16$		
Number of Top Bars in Middle Section:		$N_{TopMidBars} = 59$		
Number of Top Bars in Outer Section:		$N_{TopOutBars} = 27$		

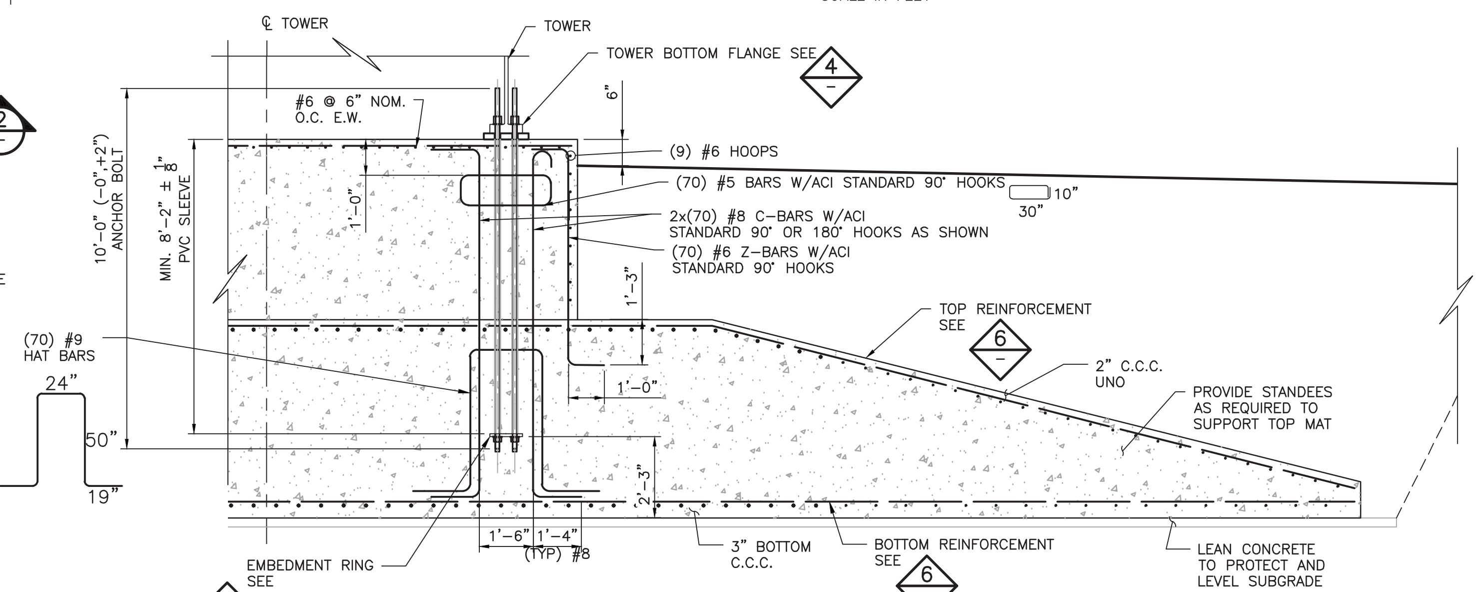
XII. Foundation Drawings and Specifications



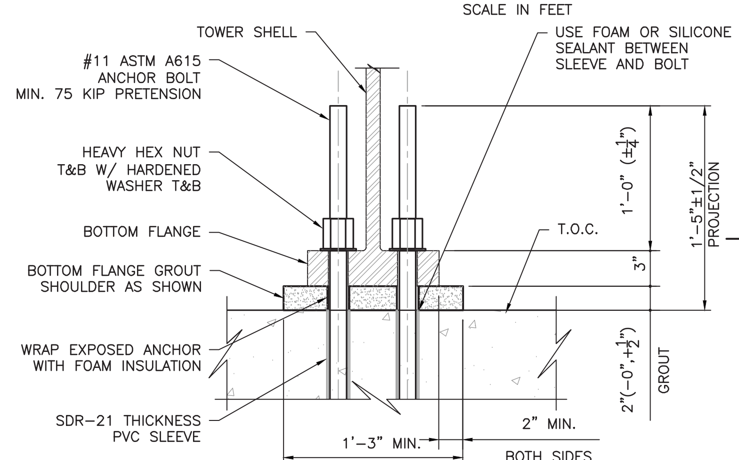
1 PLAN: TOWER FOUNDATION
3/32"=1'-0"
0 5 10 20 30
SCALE IN FEET



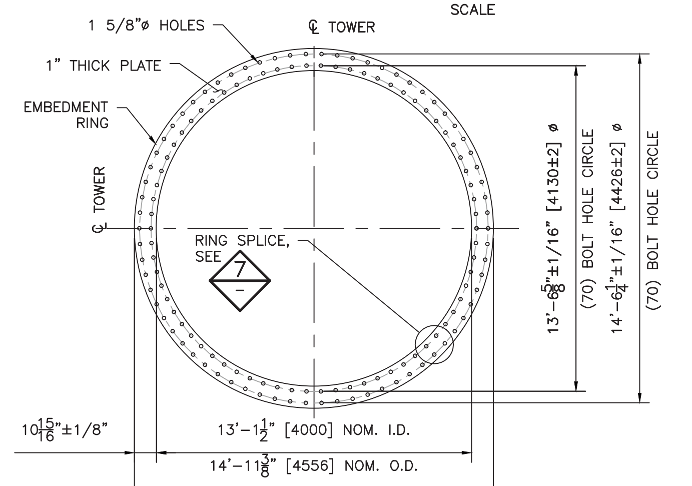
2 ELEVATION: TOWER FOUNDATION
1/8"=1'-0"
0 1 5 10
SCALE IN FEET



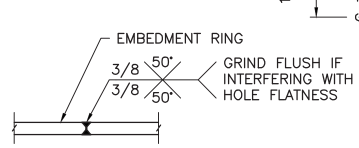
3 SECTION: TOWER FOUNDATION
3/8"=1'-0"
0 2 4 6
SCALE IN FEET



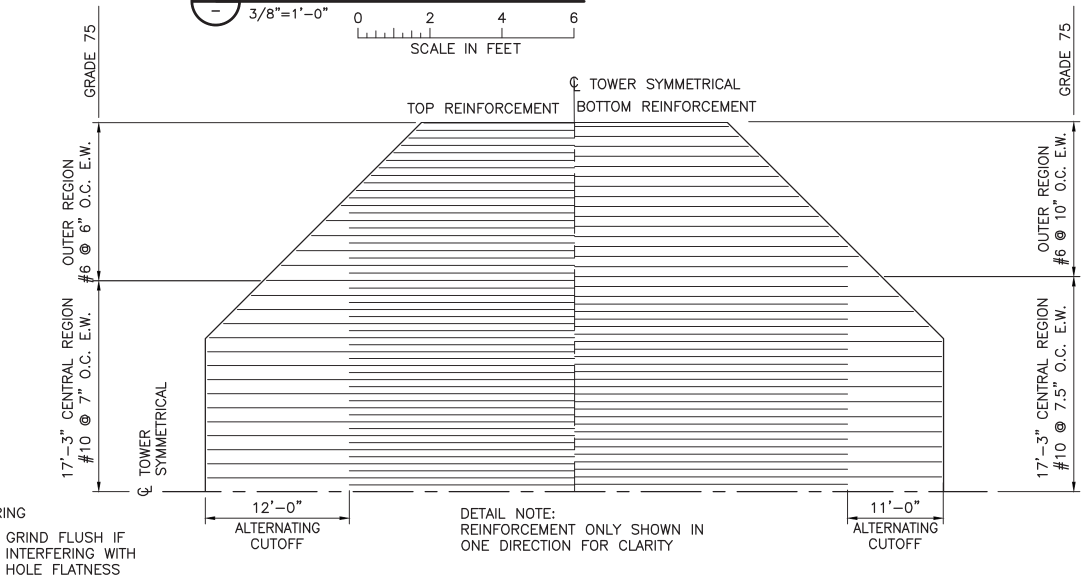
4 DETAIL: TOWER BOTTOM FLANGE
1 1/2"=1'-0"
0 3" 6" 9" 1' 2'
SCALE



5 DETAIL: EMBEDMENT RING
1/4"=1'-0"
0 1 5
SCALE IN FEET



7 DETAIL: EMBEDMENT RING SPLICE
1 1/2"=1'-0"
0 3" 6" 9" 1' 2'
SCALE



6 DETAIL: TOP & BOTTOM REINFORCEMENT
1/8"=1'-0"
0 1 5 10
SCALE IN FEET

ISSUE FOR CONSTRUCTION
CONFIDENTIAL

BUILDING AND DESIGN CODES:

INTERNATIONAL BUILDING CODE 2012, INTERNATIONAL CONFERENCE OF BUILDING OFFICIALS.

BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE, ACI 318, 2011, AMERICAN CONCRETE INSTITUTE.

WIND TURBINE AND TOWER:

MANUFACTURER: GE
MODEL: 2.5-116
POWER OUTPUT: 2.5 MW
TURBINE HUB HEIGHT: 90m
ROTOR DIAMETER: 116m

DESIGN SERVICE LOADS:

UNFACTORED SERVICE LOADS DUE TO NORMAL EXTREME WIND CONDITION IEC CLASS S
(APPLY 1.4 LOAD FACTOR TO LOADS SHOWN BELOW TO OBTAIN FACTORED LOADS)
OVERTURNING MOMENT, Mxy = 45,003 kN-m
HORIZONTAL BASE SHEAR, Hxy = 512 kN
VERTICAL TOWER LOAD, Wz = 2,699 kN

UNFACTORED SERVICE LOADS DUE TO ABNORMAL EXTREME WIND CONDITION IEC CLASS S
(APPLY 1.14 LOAD FACTOR TO LOADS SHOWN BELOW TO OBTAIN FACTORED LOADS)
OVERTURNING MOMENT, Mxy = 55,693 kN-m
HORIZONTAL BASE SHEAR, Hxy = 692 kN
VERTICAL TOWER LOAD, Wz = 2,625 kN

TOWER MISALIGNMENT LOADING, Mxy = 1,335 kN-m

FOUNDATION DESIGN DATA:

FATIGUE LIFE: 20 YEARS BASED ON FATIGUE LOADING PROVIDED IN REFERENCE 1

REFERENCE DOCUMENTS:

1. GE RENEWABLE ENERGY A/S, "FOUNDATION LOAD SPECIFICATION FOR WIND TURBINE GENERATOR SYSTEMS, STARWOOD OHIO/USA (GE PROJECT 1022819) 2.5-116 60 Hz, 90m HUB HEIGHT, COLD WEATHER EXTREME, IEC CLASS S", r01 DATED 2017-09-05.

2. BARR ENGINEERING COMPANY, "GEOTECHNICAL ENGINEERING REPORT, NORTHWEST OHIO WIND PROJECT, PAULDING COUNTY, OHIO", PREPARED FOR STARWOOD ENERGY GROUP GLOBAL, DECEMBER 2014

3. BARR ENGINEERING COMPANY, "FOUNDATION DESIGN CALCULATIONS, GE 2.5-116 90m HUB HEIGHT, NORTHWEST OHIO WIND PROJECT, PAULDING COUNTY, OHIO", PREPARED FOR WHITE CONSTRUCTION INC., DATED SEPTMEBER 2017.

MIN. 28-DAY COMPRESSIVE STRENGTH CONCRETE
5,000 PSI

MIN. YIELD POINT STRENGTH OF REINFORCING BAR:
TOP AND BOTTOM REINFORCEMENT: **75 KSI**
ALL OTHER REINFORCEMENT: **60 KSI**

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

MIN. 28-DAY COMPRESSIVE STRENGTH OF NON-SHRINK GROUT:
9,500 PSI

MIN. YIELD POINT STRENGTH OF EMBEDMENT PLATE:
36 KSI

ESTIMATED CONCRETE VOLUME:
FOOTING: **410 CUBIC YARDS**
PEDESTAL: **47 CUBIC YARDS**

ESTIMATED WEIGHT OF STEEL REINFORCING:
REINFORCEMENT:
GRADE 75: **29.1 TONS**
GRADE 60: **5.7 TONS**

COARSE AGGREGATE GRADATION:
ASTM C33 (SIZE NUMBER 6 OR 67) WITH A MINIMUM OF 2% RETAINED ON THE 3/4-INCH SIEVE.

ABBREVIATIONS:

B.O.	BOTTOM OF	O.C.	ON CENTER
C.C.C	CLEAR CONCRETE COVER	O.D.	OUTSIDE DIAMETER
℄	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	MAX.	MAXIMUM
NOM.	NOMINAL	W/	WITH
ø	DIAMETER		

STATE OF OHIO
MATTHEW B. JOHNSON
E-74181
09/15/17
REGISTERED PROFESSIONAL ENGINEER

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
A	LJH	CMM3	MBJ	7/21/17	ISSUED FOR PROCUREMENT
B	LJH	CMM3	MBJ	7/31/17	REVISED ISSUE FOR PROCUREMENT
C	LJH	CMM3	MBJ	8/10/2017	ISSUE FOR IE REVIEW
0	RRR2	CMM3	MBJ	9/15/2017	ISSUED FOR CONSTRUCTION

RELEASED TO/FOR	A	B	C	0	1	2	3
CLIENT	07/21/17	07/31/17	08/10/17				
BID							
CONSTRUCTION				09/15/17			
	DATE RELEASED						

BARR
Corporate Headquarters: Minneapolis, Minnesota Ph: 1-800-632-2277
Project Office:
BARR ENGINEERING CO.
4300 MARKETPOINTE DRIVE
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MINNEAPOLIS, MN 55435
Ph: 1-800-632-2277
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www.barr.com

Scale	AS SHOWN
Date	3/17/2017
Drawn	LJH
Checked	CMM3
Designed	LJH
Approved	MBJ

WHITE CONSTRUCTION
CLINTON, INDIANA

NORTHWEST OHIO WIND PROJECT
PAULDING COUNTY, OHIO

SPREAD FOOTING FOUNDATION
PLAN, ELEVATION, SECTION & DETAILS

BARR PROJECT No. 35631001.02
CLIENT PROJECT No.
DWG. No. S-01
REV. No. 0

1.0 GENERAL REQUIREMENTS AND SUBMITTALS

A. GENERAL
1. THE REQUIREMENTS SPECIFIED HEREIN APPLY TO THE FOLLOWING DRAWINGS:
   a. NORTHWEST OHIO WIND PROJECT, DRAWING S-01, SPREAD FOOTING FOUNDATION

B. SUBMITTALS
1. SUBMITTALS SHALL BE MADE A MINIMUM OF ONE WEEK PRIOR TO INCORPORATION INTO THE WORK. THE FOUNDATION ENGINEER (BARR) WILL REVIEW SUBMITTALS, INCLUDING THE TESTING AND INSPECTION RECORDS TO CHECK CONFORMANCE WITH THE DRAWINGS AND SPECIFICATIONS. THE REVIEW DOES NOT RELIEVE THE CONTRACTOR FROM RESPONSIBILITY FOR ERRORS IN CONSTRUCTION OF THE WORK DUE TO ERRORS CONTAINED IN THOSE DOCUMENTS.
2. SUBMIT ONE ELECTRONIC COPY OF THE SUBMITTALS SPECIFIED TO THE FOUNDATION ENGINEER AT THE FOLLOWING: BARR ENGINEERING COMPANY
ATTN: MR. CHUCK BEAUZAY [CBEAUZAY@BARR.COM]
3. SUBMIT A LIST OF THE TESTING COMPANIES THAT WILL BE UTILIZED ON THE PROJECT FOR PERFORMANCE OF TESTS SPECIFIED.
4. SUBMIT NAME AND QUALIFICATIONS OF THE GEOTECHNICAL ENGINEER.
5. SUBMIT INFORMATION (TESTING RESULTS, PRODUCT DATA, CONSTRUCTION DETAILS, ETC.) AS LISTED IN THE FOLLOWING SECTIONS: 2.B, 3.B, 4.B, 5.B, AND 6.B.

2.0 EXCAVATION, SUBGRADE PREPARATION, BACKFILL, & COMPACTION

A. GENERAL
1. COORDINATE THE EXCAVATION, SUBGRADE PREPARATION, BACKFILL, COMPACTION, AND GRADING ACTIVITIES WITH THE REFERENCED GEOTECHNICAL DOCUMENTS ON DRAWING S-01.

B. SUBMITTALS
1. SUBMIT GROUNDWATER AND SURFACE WATER CONTROL PLAN.
2. SUBMIT SUBGRADE STRENGTH AND UNIFORMITY VERIFICATION METHOD.
3. SUBMIT SUBGRADE INSPECTION REPORT FOR EACH FOUNDATION COMPLETED BY A GEOTECHNICAL ENGINEER.
4. SUBMIT GRAIN SIZE ANALYSIS PER ASTM D422, NATURAL MOISTURE CONTENT PER ASTM D2216, AND STANDARD PROCTOR MAXIMUM DRY DENSITY PER ASTM D698 FOR COMMON FILL SOIL MATERIALS.
5. SUBMIT COMPACTION TEST RESULTS FOR FILL PLACED OVER THE FOUNDATION INDICATING LOCATION OF TEST, DRY DENSITY, AND MOISTURE CONTENT OF PLACED FILL.

C. PRODUCTS
1. **LEAN CONCRETE:** CONTAINING ASTM C150, TYPE I OR ASTM C1157, TYPE GU CEMENT. COMPRESSIVE STRENGTH AND THICKNESS SHALL BE SUFFICIENT TO SUPPORT REINFORCING STEEL AND ANCHOR BOLT CAGE DURING CONSTRUCTION.
2. **COMMON FILL:** SHALL CONSIST OF SUITABLE UNFROZEN MATERIALS EXCAVATED FROM THE FOUNDATION SITE OR IMPORTED AS NECESSARY. ADDITIONAL CRUSHING AND SCREENING MAY BE REQUIRED TO PROCESS THE MATERIAL TO THE SPECIFIED REQUIREMENTS BELOW.
   a. MATERIALS BACKFILLED WITHIN 1 FOOT OF ANY CONCRETE SHALL BE FINE, WELL GRADED MATERIAL WITH PARTICLE SIZE NO GREATER THAN 3 INCHES.
   b. MATERIALS BACKFILLED BEYOND 1 FOOT OF ANY CONCRETE MAY CONSIST OF ALL OTHER EXCAVATED MATERIALS PROVIDED THEY MEET THE DENSITY REQUIREMENTS AND CAN BE PLACED USING METHODS THAT WILL PREVENT VOIDS FROM OCCURRING.
3. **ENGINEERED FILL:** CONTACT FOUNDATION ENGINEER IF NEEDED BASED ON SUBGRADE INSPECTION.

D. EXECUTION
1. CONFIRM LOCATION OF TURBINE COORDINATES IN THE REFERENCED GEOTECHNICAL DOCUMENT ON DRAWING S-01. IF TURBINE COORDINATES ARE OFFSET BY MORE THAN 50 FEET, OBTAIN WRITTEN INSTRUCTIONS FROM THE FOUNDATION ENGINEER AS TO THE MEANS OF ADDITIONAL INVESTIGATION TO BE UNDERTAKEN. OBTAIN WRITTEN CONFIRMATION FROM THE GEOTECHNICAL ENGINEER THAT THE SPECIFIED INVESTIGATION WAS COMPLETED.
2. REMOVE TOPSOIL FROM THE PLAN AREA AND STORE IN AN OWNER DESIGNATED AREA. THE TOPSOIL SHALL BE USED FOR SITE RESTORATION.
3. EXCAVATE SOILS OR ROCK TO THE LIMITS INDICATED ON DRAWING S-01 USING TECHNIQUES THAT WILL MINIMIZE DISTURBANCE TO THE SUBGRADE. CONTRACTOR SHALL BE RESPONSIBLE FOR CONTROL OF SURFACE WATER AND/OR GROUNDWATER FLOWS INTO THE EXCAVATION.
4. COMPACTION OF IN-SITU SOILS IS NOT REQUIRED WHERE CLAYEY SOILS ARE PRESENT AT THE EXCAVATION BASE. AT TURBINE SITES WHERE SANDS ARE ENCOUTERED AT THE FOUNDATION BASE ELEVATION, PERFORM SURFACE COMPACTION WITH A MINIMUM OF ONE PASS THROUGHOUT THE EXCAVATION BASE.
5. IF, IN THE COURSE OF EXCAVATING THE FOUNDATION, THE BASE OF THE EXCAVATION BECOMES RUTTED, DAMAGED OR IS OTHERWISE DETERMINED TO BE OF INADEQUATE CHARACTER, PERFORM THE FOLLOWING ACTIONS:
   a. SILTS OR CLAYS: SUBCUT THE EXCAVATION A MINIMUM OF 6 INCHES BEYOND THE DEPTH OF THE INADEQUATE SOILS AND REPLACE WITH LEAN CONCRETE OR ENGINEERED FILL. CONTACT FOUNDATION ENGINEER FOR ENGINEERED FILL REQUIREMENTS.
   b. GRANULAR SOILS: LEVEL AND SURFACE COMPACT THE EXCAVATION BASE SURFACE COMPACTION OF THIS MATERIAL TO ACHIEVE AT LEAST 98 PERCENT OF THE LABORATORY MAXIMUM DRY DENSITY MEASURED ACCORDING TO THE STANDARD PROCTOR TEST METHOD.
6. PRIOR TO PLACING PROTECTIVE LEAN CONCRETE SURFACE, HAVE A PROFESSIONAL GEOTECHNICAL ENGINEER (OR A PERSON UNDER THE GEOTECHNICAL ENGINEER'S DIRECT SUPERVISION) INSPECT THE SUBGRADE CONDITIONS AND RECORD THE SOIL TYPE ENCOUNTERED, GROUNDWATER CONDITIONS, OR OTHER SUBSURFACE CONDITIONS. A SUBGRADE INSPECTION REPORT SHALL BE PREPARED AND SUBMITTED FOR EACH FOUNDATION THAT INCLUDES THE FOLLOWING:
   a. VERIFICATION THAT OBSERVATIONS TAKEN ARE CONSISTENT WITH THE OBSERVATIONS CONTAINED IN THE REFERENCED GEOTECHNICAL DOCUMENT ON DRAWING S-01.
   b. VERIFICATION THAT SUBGRADE STRENGTH AND UNIFORMITY ARE ADEQUATE (SUBMIT FOR REVIEW THE METHODS TO BE USED TO VERIFY THE SUBGRADE STRENGTH AND UNIFORMITY).
   c. PHOTOS OF PREPARED SUBGRADE.

   IF SOIL CONDITIONS ARE ENCOUNTERED THAT ARE NOT CONSISTENT WITH THE REFERENCED GEOTECHNICAL DOCUMENTS (E.G. HALF SOILS AND HALF ROCK) OR IF SUBGRADE UNIFORMITY OR STRENGTH IS INSUFFICIENT, 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 CORRECTIVE ACTIONS WERE COMPLETED.
7. FOR PROTECTION OF THE SUBGRADE AND ESTABLISHMENT OF A WORKING SURFACE, PLACE LEAN CONCRETE FILL AS INDICATED ON DRAWING S-01. IT IS RECOMMENDED THAT THE LEAN CONCRETE FILL BE PLACED AS LEVEL AS PRACTICAL TO FACILITATE PLACEMENT OF THE REINFORCING STEEL AND EMBEDMENT RING.
8. BACKFILL AND COMPACTION: PLACE AND COMPACT COMMON FILL MATERIALS TO THE LIMITS, DEPTH AND DRY DENSITY INDICATED ON DRAWING S-01. IN ADDITION TO THE DRY DENSITY REQUIREMENT, BACKFILL MUST BE COMPACTED TO A MINIMUM OF 95% STANDARD PROCTOR. PLACE FILL IN MAXIMUM LOOSE LIFTS OF 12 INCHES OR LESS TO ACHIEVE THE SPECIFIED DENSITY. ADDITIONAL DRYING OF BACKFILL MATERIAL MAY BE NECESSARY TO ACHIEVE THESE SPECIFICATIONS. BACKFILL MAY BE PLACED WHEN THE FOOTING AND PEDESTAL HAVE REACHED 2,000 PSI.
9. GRADE THE SITE IN ACCORDANCE WITH DRAWING S-01 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
1. FOR EVERY 2500 CUBIC YARDS OF PLACED COMMON FILL, OBTAIN SAMPLES OF COMMON FILL MATERIALS AND PERFORM AND SUBMIT GRAIN SIZE ANALYSIS PER ASTM D422, MOISTURE CONTENT PER ASTM D2216, AND STANDARD PROCTOR MAXIMUM DRY DENSITY PER ASTM D698.
2. FOR ALL PLACED AND COMPACTED COMMON FILLS AROUND THE FOUNDATION, PERFORM AND SUBMIT ONE DENSITY TEST PER LIFT INDICATING TEST LOCATION, DRY DENSITY AND MOISTURE CONTENT PER ASTM D6938.
3. PROVIDE A SUBGRADE INSPECTION REPORT TO BE COMPLETED BY A GEOTECHNICAL ENGINEER FOR EACH FOUNDATION.

3.0 CAST-IN-PLACE CONCRETE AND STEEL REINFORCING

A. GENERAL
1. CONCRETE WORK SHALL BE IN COMPLIANCE WITH THE FOLLOWING CODES AND SPECIFICATIONS:
   a. ACI 301, STANDARD SPECIFICATIONS FOR STRUCTURAL CONCRETE.
   b. ACI 308, STANDARD SPECIFICATION FOR CURING CONCRETE.
   c. ACI 318 (CURRENT EDITION), BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE.
   d. ASTM C94, STANDARD SPECIFICATION FOR READY-MIX CONCRETE.
   e. ASTM C172, STANDARD PRACTICE FOR SAMPLING FRESHLY MIXED CONCRETE.
2. CONCRETE SHALL MEET THE REQUIREMENTS OF ACI 318, TABLES 19.3.1.1 AND 19.3.2.1 FOR EXPOSURE CLASSES 'F2', 'S0', 'W0', AND 'C1'.

B. SUBMITTALS
1. FOR EACH CONCRETE TYPE USED, SUBMIT FOR APPROVAL A MIX DESIGN CERTIFIED BY A PROFESSIONAL ENGINEER (LICENSED IN OHIO) AND MEETING THE MINIMUM SPECIFIED REQUIREMENTS. CONCRETE MIX SHALL BE PROPORTIONED ACCORDING TO THE REQUIREMENTS OF ACI 318, CHAPTER 5 ON THE BASIS OF FIELD DATA OR TRIAL MIXTURES.
2. SUBMIT PRODUCT DATA FOR ADMIXTURES, POZZOLAN, AND CEMENT USED ON THE PROJECT.
3. SUBMIT GRADATION, SOURCE, AND TYPE OF COARSE AND FINE AGGREGATE MEETING THE REQUIREMENTS OF ASTM C33.
4. SUBMIT REINFORCING FABRICATION AND PLACEMENT SHOP DRAWINGS.
5. SUBMIT MILL REPORTS OF REINFORCING STEEL, CONFIRMING THE GRADE AND STRENGTH OF REINFORCING STEEL PROVIDED ON THE PROJECT.
6. SUBMIT QUALITY CONTROL FIELD TESTS OF AIR CONTENT, SLUMP, AIR TEMPERATURE, AND CONCRETE TEMPERATURE.
7. SUBMIT CONCRETE CYLINDER STRENGTH TEST RESULTS.
8. SUBMIT A PLAN FOR HOT AND COLD WEATHER PROTECTION OF CONCRETE IN ACCORDANCE WITH ACI 305-306.
9. SUBMIT A PLAN FOR CONCRETE CURING IN ACCORDANCE WITH ACI 308.
10. ASR REQUIREMENTS: IF AGGREGATES CONTAIN POTENTIALLY REACTIVE MATERIALS (AS DETERMINED BY ONE OF THE TEST METHODS OUTLINED IN ASTM C33, APPENDIX X1), SUBMIT TEST RESULTS INDICATING THE POTENTIAL REACTIVITY, SUCH AS THE RESULTS OF TESTING TO ASTM C295, C289, C1293, OR C1260. IF THESE TEST RESULTS INDICATE THE AGGREGATES ARE REACTIVE, SUBMIT AN ASR MITIGATION PLAN, INCLUDING VERIFICATION THAT THE PROPOSED MEASURES WILL SUFFICIENTLY LIMIT ASR TO PREVENT EXCESSIVE EXPANSION. THIS VERIFICATION SHALL CONSIST OF THE RESULTS OF TESTS PERFORMED ACCORDING TO ASTM C1567, AASHTO T303, OR ASTM C1293.
11. SUBMIT FOR APPROVAL A MASS CONCRETE PLACEMENT AND TEMPERATURE CONTROL PLAN MEETING THE REQUIREMENTS OF ACI 301 CHAPTER 8 AND ACI 207.1R.

C. PRODUCTS
1. **REINFORCING BARS:** TO ASTM A615, GRADE 60 OR GRADE 75 AS NOTED ON DRAWING S-01, DEFORMED, UNCOATED.
2. **CEMENT:** TO ASTM C150, TYPE I, OR ASTM C1157, TYPE GU.
3. **FLY ASH:** TO ASTM C618, CLASS C OR F (IF SPECIFIED).
4. **MINIMUM CEMENTITIOUS CONTENT:** IN ACCORDANCE WITH APPROVED MIX DESIGN.
5. **COARSE AND FINE AGGREGATES:** TO ASTM C33, GRADATION IN ACCORDANCE WITH SPECIFICATIONS AND APPROVED MIX DESIGN. NOMINAL MAXIMUM AGGREGATE SIZE SHALL BE AS SHOWN ON DRAWING S-01. ALL AGGREGATES MUST BE NON-REACTIVE WITH CEMENT TO PREVENT ASR.
6. **AIR ADMIXTURE AND CONTENT:** TO ASTM C260, 6% FOR PEDESTAL ONLY, NO AIR CONTENT REQUIREMENT FOR FOOTING.
7. **OTHER ADMIXTURES:** CHLORIDE FREE WATER REDUCING ADMIXTURE AND SUPERPLASTICIZER AS REQUIRED.
8. **MAXIMUM WATER CEMENT RATIO:** 0.45.
9. **28 DAY COMPRESSIVE STRENGTH:** 5,000 PSI.
10. **SLUMP:** IN ACCORDANCE WITH APPROVED MIX DESIGN AT THE POINT OF DEPOSITION WITH THE ADDITION OF ADMIXTURES.
11. **CONCRETE UNIT WEIGHT:** 145 PCF (MINIMUM) TO ASTM C138.

D. EXECUTION
1. PLACE CONCRETE AND REINFORCING AS SHOWN AND IN ACCORDANCE WITH THE FOLLOWING TOLERANCES:
   a. REINFORCING PLAN SPACING: PLUS OR MINUS 2 INCHES.
   b. REINFORCING VERTICAL SPACING: PLUS OR MINUS 1 INCH.
   c. FOOTING CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 3 INCHES.
   d. PEDESTAL CLEAR CONCRETE COVER: MINUS 0 INCHES, PLUS 2 INCHES.
   e. FOOTING PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 3 INCHES.
   f. FOOTING THICKNESS: MINUS 0 INCHES, PLUS 3 INCHES.
   g. PEDESTAL PLAN DIMENSIONS: MINUS 0 INCHES, PLUS 2 INCHES.
   h. PEDESTAL HEIGHT: MINUS 1 INCH, PLUS 0 INCHES.
   i. PEDESTAL CENTERED TO WITHIN 2 INCHES RELATIVE TO FOOTING.
   j. CONCRETE AIR CONTENT: +/- 1.5%
2. PROVIDE NECESSARY TIES, CHAIRS, AND STANDEES TO SECURE AND SUPPORT REBAR AND PREVENT PERMANENT DISPLACEMENT OR MOVEMENT OF THE BARS GREATER THAN 1 INCH DURING PLACEMENT OF CONCRETE. REBAR THAT DEFLECTS BUT RETURNS TO ITS ORIGINAL POSITION IS ACCEPTABLE.
3. REINFORCEMENT SHALL BE FREE OF LOOSE RUST, MILL SCALE, EARTH, ICE, CONCRETE, OR OTHER MATERIALS WHICH COULD PREVENT BONDING TO NEW CONCRETE.
4. SET FORMWORK PER ACI 347 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.
5. PLACE CONCRETE IN ACCORDANCE WITH ACI 318. PLACE SUCCESSIVE LIFTS OF CONCRETE AS QUICKLY AS POSSIBLE TO ENSURE PROPER AMALGAMATION OF CONCRETE BETWEEN SUCCESSIVE LIFTS.
6. CONSOLIDATE CONCRETE IN ACCORDANCE WITH ACI 318 PREVENTING THE FORMATION OF JOINTS, VOIDS, HONEYCOMBING OR SEGREGATION OF AGGREGATE.
7. ROUGH FINISH TOP OF CONCRETE FOOTING USING A ROLLER SCREED.
8. PRIOR TO PLACING PEDESTAL CONCRETE, CLEAN CONCRETE SURFACE WITH AIR OR WATER TO REMOVE DEBRIS AND OTHER LOOSE MATERIAL FROM TOP OF FOOTING.
9. TROWEL AND BROOM FINISH TOP OF PEDESTAL.
10. CURE CONCRETE FOOTING AND PEDESTAL IN ACCORDANCE WITH ACI 318 AND 308. IF A CURING MEMBRANE IS USED, APPLY CURING MEMBRANE AS SOON AS BLEEDING HAS STOPPED AND FREE WATER HAS DISAPPEARED FROM THE SURFACE.
11. 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.
12. ALL HOOKS SHOWN ON REBAR SHALL BE STANDARD HOOKS (UNO).
13. ANY SHRINKAGE CRACKS IN EXCESS OF 0.012 INCHES (0.3mm) IN WIDTH SHALL BE SEALED WITH AN ENGINEER APPROVED PRODUCT.
14. JOBSITE ADDITION OF WATER TO AIR ENTRAINED CONCRETE IS PROHIBITED.
15. MONITOR MASS CONCRETE TEMPERATURES IN ACCORDANCE WITH THE MASS CONCRETE TEMPERATURE CONTROL PLAN.

E. TESTING AND INSPECTION
1. FOR EACH FOOTING PLACED, CAST A MINIMUM OF (2) 6-INCH OR (3) 4-INCH DIAMETER CONCRETE CYLINDERS PER ASTM C31 FOR EVERY 150 CUBIC YARDS, OR FRACTION THEREOF, OF CONCRETE PLACED FOR LABORATORY STRENGTH TESTING PER ASTM C39. PERFORM ONE "STRENGTH TEST" AT 28 DAYS FOR EVERY 150 CUBIC YARDS, OR FRACTION THEREOF, OF CONCRETE PLACED ("STRENGTH TEST" = AVERAGE OF (2) 6-INCH OR (3) 4-INCH CYLINDER BREAKS). FOR EACH FOOTING PLACED, CAST (2) 6-INCH OR (3) 4-INCH ADDITIONAL CONCRETE CYLINDERS PER ASTM C31, AND IF NECESSARY PERFORM ONE "STRENGTH TEST" PER ASTM C39 AT 56 DAYS. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
2. FOR EACH PEDESTAL, CAST A MINIMUM OF (4) 6-INCH OR (6) 4-INCH DIAMETER CONCRETE CYLINDERS PER ASTM C31 FOR LABORATORY STRENGTH TESTING PER ASTM C39. PERFORM ONE "STRENGTH TEST" AT 28 DAYS ("STRENGTH TEST" = AVERAGE OF (2) 6-INCH OR (3) 4-INCH CYLINDER BREAKS) AND IF NECESSARY ONE AT 56 DAYS. CAST ADDITIONAL CYLINDERS AS REQUIRED TO DETERMINE CONCRETE STRENGTH AT OTHER TIMES.
3. PERFORM A MINIMUM OF ONE AIR TEST PER ASTM C231 AND A MINIMUM OF ONE SLUMP TEST PER ASTM C143 PER SET OF CYLINDERS CAST. RECORD AMBIENT AIR TEMPERATURE AND CONCRETE TEMPERATURE PER ASTM C1064.
4. PERFORM TESTING AND INSPECTION REQUIRED BY THE MASS CONCRETE TEMPERATURE CONTROL PLAN.

4.0 ANCHOR BOLTS AND EMBEDMENT RING

A. GENERAL
1. PRODUCTS, SUBMITTALS, EXECUTION, AND TESTING ARE SPECIFIED TO PROVIDE DURABLE ANCHOR BOLTS AND EMBEDMENT PLATES.

B. SUBMITTALS
1. SUBMIT PRODUCT DATA AND SHOP DRAWING FOR ANCHORS AND HARDWARE.
2. SUBMIT A 12-INCH LONG PRODUCT SAMPLE OF THE ANCHOR COMPLETE WITH WASHER AND NUT.
3. SUBMIT MILL CERTIFICATES FOR ANCHORS INDICATING YIELD AND TENSILE STRENGTH OF ANCHORS.
4. SUBMIT MILL CERTIFICATES FOR THE EMBEDMENT RING INDICATING THAT THE MATERIAL MEETS THE MINIMUM STRENGTH REQUIREMENTS.
5. SUBMIT LABORATORY TENSION TESTS OF ANCHOR COMPLETE WITH THREADS.
6. SUBMIT A TENSIONING CALIBRATION PROCEDURE FOR REVIEW, INCLUDING VERIFICATION THAT THE EQUIPMENT PROVIDED AND TENSIONING METHODS USED ARE DELIVERING THE NECESSARY LOCK OFF LOAD.
7. SUBMIT A TENSIONING PROCEDURE FOR REVIEW.
8. SUBMIT A TENSION TESTING PROCEDURE FOR REVIEW.
9. SUBMIT TENSION TEST DATA FOR ANCHOR BOLTS THAT ARE TESTED INDICATING BOLT LOCATION AND TENSION VALUE.
10. SUBMIT EMBEDMENT RING AND TEMPLATE RING SHOP DRAWINGS.

C. PRODUCTS
1. **ANCHOR BOLTS:** #11 SIZE WITH MATERIAL TO ASTM A615 GRADE 75, WITH COLD ROLLED THREADS, A MINIMUM YIELD STRENGTH OF 75 KSI, A MINIMUM TENSILE STRENGTH OF 100 KSI, A MAXIMUM THREAD DIAMETER OF 1.50 INCHES, AND A MINIMUM NET AREA OF 1.56 SQUARE INCHES.
2. **ANCHOR BOLT SLEEVES:** TO ANCHOR BOLT MANUFACTURER'S REQUIREMENTS.
3. **EMBEDMENT RING:** TO ASTM A36, PLAIN FINISH. NEW MATERIAL (NO REUSED TEMPLATES).
4. **HEAVY HEX NUTS:** TO ANCHOR BOLT MANUFACTURER'S SPECIFICATIONS. NUTS SHALL BE CAPABLE OF DEVELOPING THE MINIMUM TENSILE STRENGTH OF THE ANCHOR.
5. **HARDENED STEEL WASHERS:** TO ASTM F436, PLAIN FINISH.

D. EXECUTION
1. THE FOLLOWING TOLERANCES SHALL BE ADHERED TO FOR PLACEMENT OF ANCHOR BOLTS:
   a. ANCHOR BOLT PLAN LOCATION - PLUS OR MINUS 1/16 INCH.
   b. ANCHOR BOLT PLUMBNESS - LESS THAN 1/4 DEGREE.
   c. TEMPLATE AND EMBEDMENT RING PLAN DIMENSION - PLUS OR MINUS 1/16 INCH.
   d. EMBEDMENT RING LEVEL - PLUS OR MINUS 1/4 INCH.
   e. EMBEDMENT RING ELEVATION - PLUS OR MINUS 1/2 INCH.
2. THE BOTTOM OF THE ANCHOR BOLT SHALL EXTEND BEYOND THE BOTTOM NUT BY A MINIMUM OF 1/2 INCH.
3. USE A TEMPLATE RING TO SET ANCHOR BOLT PLUMBNESS AND POSITION. ENSURE THE TEMPLATE RING IS SET IN ACCORDANCE WITH THE SPECIFIED CONSTRUCTION TOLERANCES.
4. 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.
5. 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.
6. AFTER SETTING AND GROUTING OF THE LOWER TOWER SECTION(S) AND AFTER THE CONCRETE AND GROUT HAS ACHIEVED THE REQUIRED STRENGTH GIVEN IN SECTION 7.0, USE AN APPROVED TENSIONING PROCEDURE TO APPLY A LOCK-OFF FORCE TO EACH ANCHOR BOLT WHICH IS NO GREATER THAN 8 KIPS MORE THAN THE SPECIFIED TENSION FORCE. 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 TO ENSURE REQUIRED TENSIONS ARE ACHIEVED.

E. TESTING AND INSPECTION
1. SUBMIT 3 LABORATORY TENSION TESTS FOR ANCHOR BOLTS FOR EACH HEAT NUMBER FURNISHED, COMPLETE WITH THREADS, PERFORMED BY AN INDEPENDENT TESTING LABORATORY. PERFORM TEST IN ACCORDANCE WITH ASTM A370, AND REPORT YIELD STRESS AND TENSILE STRESS.
2. AFTER ALL BOLTS HAVE BEEN TENSIONED, A MINIMUM OF 10% OF THE TOTAL BOLTS INSTALLED PER FOUNDATION SHALL BE RANDOMLY TESTED TO VERIFY THAT THE SPECIFIED 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.

5.0 TOWER BASE GROUT

A. GENERAL
1. COORDINATE GROUTING PROCEDURES WITH THE REQUIREMENTS OF THE TOWER MANUFACTURER.

B. SUBMITTALS
1. SUBMIT MANUFACTURER'S GROUT PRODUCT DATA AND MANUFACTURER'S APPROVED MIXING, PLACING AND CURING INSTRUCTIONS FOR GROUT TO BE PLACED.
2. SUBMIT GROUT CUBE STRENGTH TEST RESULTS.
3. SUBMIT CONTRACTOR'S TOWER BASE SETTING/GROUTING PLAN.

C. PRODUCTS
1. **EPOXY NON-SHRINK GROUT:** PREPACKAGED EPOXY GROUT WITH A MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS ACCORDING TO ASTM C579 AS SHOWN ON DRAWING S-01 AND A MAXIMUM COEFFICIENT OF THERMAL EXPANSION OF 30 X 10-6 IN/IN/°F IN ACCORDANCE WITH ASTM C531.
2. **CEMENTITIOUS NON-SHRINK GROUT:** PREPACKAGED GROUT CONFORMING TO ASTM C1107, WITH A MINIMUM COMPRESSIVE STRENGTH AFTER 28 DAYS ACCORDING TO ASTM C109, AS SHOWN ON DRAWING S-01.

D. EXECUTION
1. MIX, PLACE, AND CURE GROUT IN ACCORDANCE WITH APPROVED MANUFACTURER'S INSTRUCTIONS.
2. FOR CEMENT GROUTS, PROVIDE GROUT SHOULDERS IN ACCORDANCE WITH DRAWING DETAILS. DO NOT ALLOW GROUT TO BE PLACED AGAINST THE SIDE OF THE TOWER FLANGE.
3. FOR EPOXY GROUTS, POUR GROUT ACCORDING TO THE MANUFACTURER'S RECOMMENDATIONS. IF GROUT IS PLACED UP THE SIDE OF THE TOWER FLANGE, PROVIDE A 1/4 INCH EXPANSION JOINT BETWEEN THE TOWER FLANGE AND THE GROUT, AND SEAL EXPANSION JOINT WITH AN APPROVED SEALANT.
4. ONCE GROUT CUBES ARE MOLDED IN THE FIELD THEY SHALL REMAIN UNDISTURBED AND PROTECTED FROM EXTREMES IN TEMPERATURE AND VIBRATION AT THE PROJECT SITE FOR AT LEAST 18 HOURS.

E. TESTING
1. CAST MINIMUM OF 9 GROUT CUBES FOR EACH FOUNDATION.
2. PERFORM TWO LABORATORY "GROUT STRENGTH TESTS" PER ASTM C109 AT 28 DAYS ("GROUT STRENGTH TEST" = AVERAGE OF THREE CUBE BREAKS) AND IF NECESSARY ONE AT A LATER DATE. CAST ADDITIONAL GROUT CUBES AS REQUIRED TO DETERMINE STRENGTH AT OTHER TIMES.

6.0 MISCELLANEOUS CONCRETE EMBEDMENTS

A. GENERAL
1. COORDINATE THE LOCATION AND PLACEMENT OF GROUNDING GRIDS, CONTROL CONDUIT AND ELECTRICAL CONDUIT.

B. SUBMITTALS
1. 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
1. NO ITEMS.

D. EXECUTION
1. VERIFY THE LOCATION OF MISCELLANEOUS CONCRETE EMBEDMENTS AND CONDUIT SO AS NOT TO INTERFERE WITH THE FOUNDATION'S STRUCTURAL REINFORCING STEEL.
2. ENSURE THAT MISCELLANEOUS EMBEDMENTS ARE PROPERLY SECURED TO PREVENT MOVEMENT DURING CONCRETE PLACEMENT.
3. TOP OF CONDUIT MUST BE A MINIMUM OF 24 INCHES BELOW TOP OF PEDESTAL.

7.0 TOWER ERECTION AND ANCHOR TENSIONING REQUIREMENTS

A. GENERAL
1. TOWER SECTIONS MAY BE ERECTED, LEVELED AND GROUTED IN ACCORDANCE WITH SUBMITTAL 5.B.3 ABOVE.
2. ANCHORS MAY BE TENSIONED WHEN:
   a. THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED 5,000 PSI.
   b. THE GROUT STRENGTH HAS REACHED 5,000 PSI.
3. THE NACELLE AND BLADES MAY BE ERECTED WHEN:
   a. THE CONCRETE STRENGTH OF THE FOOTING AND PEDESTAL HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
   b. THE GROUT STRENGTH HAS REACHED THE SPECIFIED 28 DAY STRENGTH.
   c. UPON COMPLETION OF THE ANCHOR BOLT TENSIONING AND TESTING AS FOUND IN SECTION 4.E.2 VERIFYING THAT THE REQUIRED TENSION VALUE HAS BEEN ACHIEVED.

STATE OF OHIO
MATTHEW B. JOHNSON
E-74181
09/15/17
REGISTERED PROFESSIONAL ENGINEER

ISSUE FOR CONSTRUCTION
CONFIDENTIAL

NO.	BY	CHK.	APP.	DATE	REVISION DESCRIPTION
A	KLT	CPB	CMM3	7/28/17	ISSUED FOR PROCUREMENT
0	RRR2	CMM3	MBJ	9/15/17	ISSUE FOR CONSTRUCTION

RELEASED TO/FOR	A	B	C	0	1	2	3
CLIENT	07/28/17						
BID							
CONSTRUCTION				09/15/17			
DATE RELEASED							

BARR
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Scale	NONE
Date	3/17/2017
Drawn	KLT
Checked	CMM3
Designed	CPB
Approved	CMM3

WHITE CONSTRUCTION
CLINTON, INDIANA

NORTHWEST OHIO WIND PROJECT
PAULDING COUNTY, OHIO

SPREAD FOOTING FOUNDATION
TECHNICAL SPECIFICATIONS AND SUBMITTALS

BARR PROJECT No. 35631001.02
CLIENT PROJECT No. –
DWG. No. S-02
REV. No. 0

cpb M:\AdeptWork\cpb\Wind Template S-02 Standard Specification-Updated.dwg Plot at 0 03/16/2017 12:55:07

XIII. GE Reference

GE Renewable Energy

Foundation Load Specification for Wind Turbine Generator Systems

Starwood
Ohio / USA
(GE Project 1022819)
2.5-116
60 Hz

90m Hub Height
Cold Weather Extreme
IEC Class S

REVISION HISTORY

Rev	Release Date	Affected Pages	Change
01	2017-09-05	All	Initial Issue - Issued "For Construction"

Table of Contents

1 Loads for Foundation Design

The following loads include inertia, mass and aerodynamic forces acting on the rotor and hub. They also include forces caused by accelerations or other dynamic reactions. Partial safety factors for the loads have been applied. All additional safety factors (e.g. on materials, uncertainty of calculation method, etc.) have to be applied according to the regulations. The loads in this document for the foundation design are calculated with a full dynamic simulation program called Flex 5. The loads are given in the coordinate system shown in Figure 1. **The extreme loads for the hub height of 90m are given at a height h= 0.745m above the grade elevation from which the nominal hub height is defined. The loads have to be extrapolated to the elevation of the foundation under design consideration.**

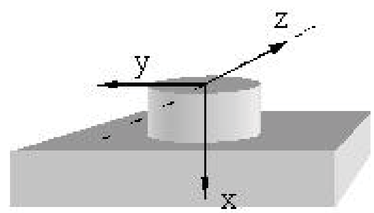

Figure 1: Coordinate System

All loads are shown with and without partial safety factors in the tables and they are directly ready for use in the foundation engineer's calculations. In addition, imperfections due to a misalignment of the tower of 8 mm/m have to be considered as per Section 3.0.

1.1 Extreme Loads

The following tables show controlling load cases (with and without partial safety factors), with all of the conditions that comprise the load case occurring simultaneously. The foundation has to be designed according to the specific country regulations. The proper final design values of partial safety factors for material properties and country specific minimum partial safety factors on loads must also be applied to these loads.

γ_F = partial safety factor for load factor design as required per International Electrotechnical Commission (IEC).

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ [-]
DLC 7.1	**2905.8**	54.9	323.2	-3000.8	19611.8	-3488.8	327.8	19919.3	1.00
DLC 6.2	2634.4	**-695.0**	-57.1	-1833.3	-9013.7	54820.8	697.3	55558.4	1.00
DLC 2.2	2674.9	-4.6	**595.5**	-549.9	50244.9	1527.4	595.5	50268.0	1.00
DLC 2.2	2716.2	-260.5	354.8	**-6454.6**	29724.2	24683.0	440.2	38636.9	1.00
DLC 2.3	2681.1	-30.9	582.1	-15.9	**50701.5**	4187.4	582.9	50874.6	1.00
DLC 6.2	2624.6	-690.9	29.8	-2225.6	-3074.7	**55608.0**	691.6	55692.7	1.00
DLC 6.2	2634.4	-695.0	-57.1	-1833.3	-9013.7	54820.8	**697.3**	55558.4	1.00
DLC 6.2	2624.6	-690.9	29.8	-2225.6	-3074.7	55608.0	691.6	**55692.7**	1.00

Table 1: Extreme loads; excluding partial safety factor

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ [-]
DLC 2.1	**3781.7**	-166.5	203.9	-4941.4	10842.9	14600.7	263.2	18186.5	1.35
DLC 6.2	2897.9	**-764.5**	-62.8	-2016.7	-9915.1	60302.9	767.1	61112.5	1.10
DLC 1.3	3618.9	-31.8	**741.7**	-436.8	57446.8	5575.0	742.4	57716.7	1.35
DLC 2.2	2987.8	-286.5	390.3	**-7100.1**	32696.7	27151.3	484.2	42500.2	1.10
DLC 1.3	3643.4	-55.8	689.4	-375.3	**60454.9**	6017.5	691.7	60753.7	1.35
DLC 6.2	2887.1	-760.0	32.7	-2448.2	-3382.2	**61168.8**	760.7	61262.32	1.10
DLC 6.2	2897.9	-764.5	-62.8	-2016.7	-9915.1	60302.9	**767.1**	61112.5	1.10
DLC 6.2	2887.1	-760.0	32.7	-2448.2	-3382.2	61168.8	760.7	**61262.32**	1.10

Table 2: Extreme loads; including partial safety factor

1.2 Load Case for Check against Lift-off

To ensure proper foundation stiffness during operation, the foundation is not allowed to lift-off of the subsoil for the following loads. These loads are provided at the tower base T-flange and the check has to be done with the loads extrapolated to the foundation bottom.

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 1.0	2708.7	28.6	409.6	912.0	35094.1	5202.5	410.6	35477.7	1.00

Table 3: Load cases for check against foundation lift-off

1.3 Load Case for Check against Overturning

To ensure the stability of the foundation, the foundation is only allowed to lift-off up to its centerline for the following load cases. These loads are provided at the tower base T-flange and the check has to be done with the loads extrapolated to the foundation bottom.

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 6.2	2624.6	-690.9	29.8	-2225.6	-3074.7	55608.0	691.6	**55692.7**	1.00

Table 4: Load case for check against overturning

1.4 Load Case for Check against Sliding

To ensure the stability of the foundation, the foundation is not allowed to slide for the following load cases. These loads are provided at the tower base T- flange and the check has to be done with the loads extrapolated to the foundation bottom.

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 2.2	2716.2	-260.5	354.8	**-6454.6**	29724.2	24683.0	440.2	38636.9	1.00
DLC 6.2	2634.4	-695.0	-57.1	-1833.3	-9013.7	54820.8	**697.3**	55558.4	1.00

Table 5: Load case for check against sliding

1.5 Load Case for Check against Shear Failure

To ensure the stability of the foundation, the foundation has to be checked of shear failure for the soil specified in the geotechnical report. These loads are provided at the tower base T-flange and the check has to be done with the loads extrapolated to the foundation bottom.

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 6.2	2887.1	-760.0	32.7	-2448.2	-3382.2	61168.8	760.7	**61262.32**	1.10

Table 6: Load case for check against shear failure

1.6 Load Case for Check against Pile Tension

No tension loading is allowed in the piles for the following load combination, unless dynamic and fatigue loading is explicitly considered in the design of the piles, including all dynamic soil-pile interaction effects. These loads are provided at the tower base T-flange:

Load case	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	Fr [kN]	Mr [kNm]	γ_F [-]
DLC 1.3	2722.6	74.0	456.5	1712.3	38372.2	9178.0	462.4	39454.6	1.00

Table 7: Load cases for check against pile tension loading

1.7 Earthquake Loads

Site specific seismic loads at the base of the tower structure can be provided by GE upon request. The request should include the applicable code, and the site specific seismic parameters (e.g., design peak ground acceleration (PGA) or equivalent as per the code, soil type, etc.). Given the level of seismicity at the site of the Starwood wind farm project, seismic loads are expected to not govern the design of the foundation at this site.

1.8 Fatigue Loads

The fatigue loads that result from the operation of the turbine are given as load spectra at the tower base T-flange. The partial safety factor on loads included in the fatigue spectra is 1.0. The combined safety factors (including partial safety factors on loads and material) on these loads to be used are:

- Fatigue check of concrete according to CEB-FIB Model Code 1990:
  - $\gamma_F \cdot \gamma_{Sd} \cdot \gamma_C$ = **1.65**.
- Fatigue check of reinforcement bars acc. to CEB-FIB Model Code 1990:
  - $\gamma_F \cdot \gamma_{Sd} \cdot \gamma_C$ = **1.265**.
- Fatigue check of embedded steel parts acc. to Eurocode 3 and IEC 61400:
  - $\gamma_F \cdot \gamma_M$ = **1.265**.
- Fatigue check of embedded steel parts acc. to Eurocode 3 and DIBt-Guidelines:
  - $\gamma_F \cdot \gamma_M$ = **1.25**.

1.9 Load Spectra Procedure with a Constant Mean Value

The load spectra are provided separately in the following document. The unit of force is kN and the unit of moment is kNm for the values provided in the file.

Filename: Load_Spectra_1022819_Starwood_2.5-116_90mHH_r01.xlsx

Mean loads at rated wind speed (constant for all load cycles)

Load at Tower Base	Fx [kN]	Fy [kN]	Fz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]	γ_F [-]
	2689.0	-5.3	344.1	79.3	30130.0	2445.0	1.00

Table 8: Mean loads at rated wind speed (for use with the fatigue load spectra files

1.10 Markov Matrices Procedure

The Markov Matrices files are provided separately in the following document. The unit of force is kN and the unit of moment is kNm for the values provided in the file.

Filename: Markov_Matrices_1022819_Starwood_2.5-116_90mHH_r01.xlsx

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Loads at Tower Base	My [kNm]	Mx [kNm]	Fz [kN]	Fx [kN]
	MY_0405	MX_0404	FZ_0403	FX_0401

Table 9: Fatigue loads (Markov Matrices)

2 Dynamic Stiffness of the Foundation

The minimum values for the dynamic foundation stiffness that have to be achieved are:

$$k_{\varphi,min} = 5.0 \cdot 10^7 \text{ kNm/rad}; \quad k_{yz,min} = 1.0 \cdot 10^6 \text{ kN/m}$$

The minimum value for the static foundation stiffness that has to be achieved is 1/5 of the dynamic stiffness:

$$k_{\varphi,stat,min} = 1.0 \cdot 10^7 \text{ kNm/rad}$$

These values are for spread (raft) type foundations or mat plus deep piling type systems only. For the special case of short pole type foundations consult GE for the specific stiffness requirements. Short pole type foundation means drilled single shaft, monopile, caisson, bored piles or the proprietary design-"Patrick and Henderson Foundation" systems.

3 Maximum Allowed Inclination for Additional Load Consideration

Maximum allowed inclination caused by non-uniform settlement of the foundation, inaccurateness of installation, and tower axis misalignment:

- Uneven settlement due to non uniform soil properties across the foundation: 3mm/m (0.17°)
- Inaccurate installation: 3mm/m (0.17°)
- Tower axis misalignment due to solar irradiation: 2mm/m(0.11°)

To account for the impact from the total misalignment of 8 mm/m on the foundation design an additional moment of 1335 kNm has to be added at the foundation upper edge with appropriate partial safety factors.

4 Connection between Tower and Foundation

The connection between tower and foundation is established with a grouted joint. The tower base flange must be in full contact with the grout; no ring plate or other similar element (except for shimming provisions) shall be placed between the tower base flange and the underlying grout.

The tower base flange anchorage consists of **140 anchor bolts – M39 of grade 8.8**. Figure 2 shows the tower base flange geometry.

M39 refers to the final nominal outer diameter of the cold formed (mechanically rolled) anchor threads. There shall be a minimum diametrical clearance of 3mm between the tower flange hole and both the threaded and unthreaded length of the anchor bolt (for anchor bolts with mechanically rolled threads, the minimum diametrical clearance is typically dictated by the nominal outer thread diameter). Use of imperial anchor sizes is permissible.

Refer to Reference [1] for recommendations and requirements on proper anchor bolt fabrication and placement, including corrosion protection, providing sufficient anchor bolt projection for engagement of the anchor bolt tensioning device and protection of anchor bolt threads during concrete and grout placement.

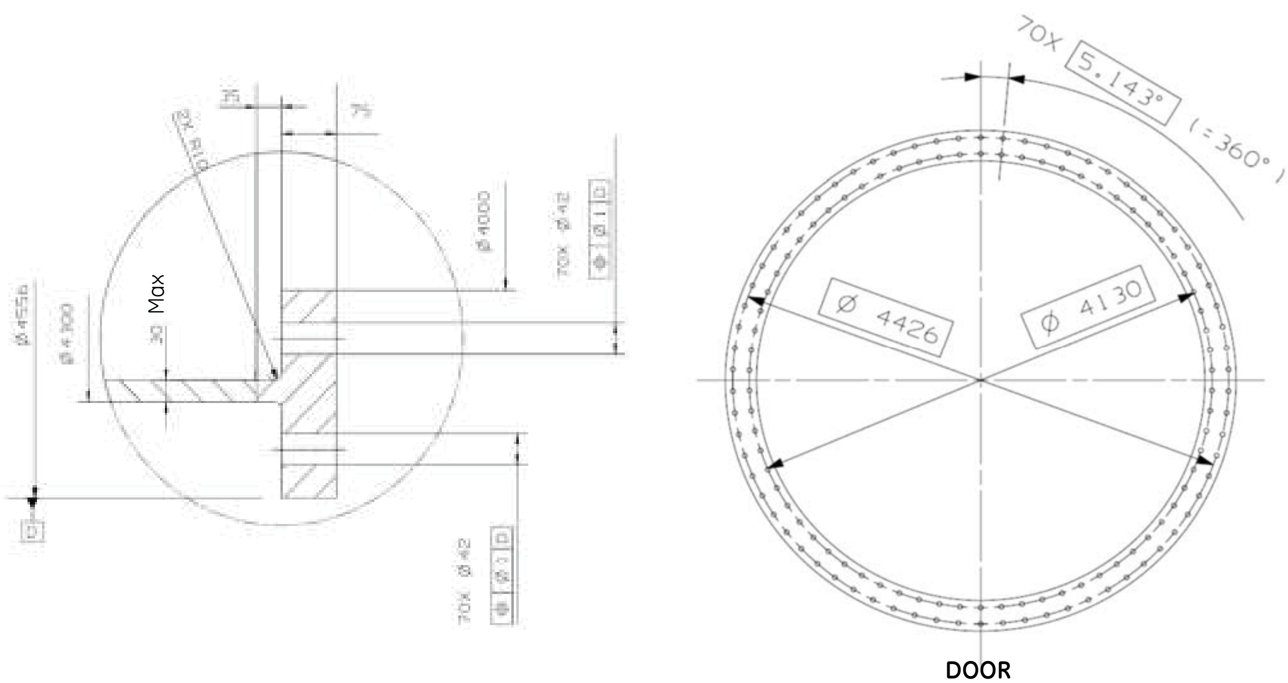

Figure 2: Base Flange Geometry Tower

5 Additional Information

Refer to Reference [1] for information on the foundation design, detailing, and execution including conduit and grounding details, foundation boundary conditions, subsoil properties, foundation spring constants, a foundation design check list, etc.

6 References

[1] GE Document "*Foundation_General_Information_Tubular_Towers_Generic_xxHz_EN_r01*", Information on the Design, Detailing and Execution of the Foundation for On-Shore Wind Turbines with Tubular Steel Towers