INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

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TECHNICAL MEMORANDUM

TOWER FOUNDATION CAP

DOCUMENT NUMBER: TM1002

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Prepared For: Texas Department of Transportation

TxDOT Bridge Division
118 E. Riverside Drive
Austin, TX 78704
Contact: Jamie Farris, PE

Prepared By: SYSTRA INTERNATIONAL BRIDGE TECHNOLOGIES

9325 Sky Park Court Suite 320
San Diego, CA 92123

STATE OF TEXAS
MICHAEL W. SMART
113501
LICENSED
PROFESSIONAL ENGINEER

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ORIGINATORS: Erwan Allanic, PE (CA) and Michael W. Smart, PE

Revision History

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1. Introduction

This technical memorandum discusses previously reported findings of the Independent Structural Analysis (ISA) concerning the tower foundation caps of the Corpus Christi New Harbor Bridge, cable-stay main bridge. The information presented herein demonstrates that the tower foundation caps do not meet the project requirements for resisting flexure and shear during design wind events.

These findings have not yet been addressed by the Developer. Also, these findings were not addressed by changes in wind input in the recently received Rev. 2 Wind Report (see Reference 3. below).

The main body of this memorandum provides context and discusses this finding. Relevant supporting calculations are included in the Appendices.

2. References

The following documents are referenced in this memorandum.

1. American Association of State Highway and Transportation Officials (AASHTO), "LRFD Bridge Design Specifications," 7th Edition, 2014 with 2015 Interim Revisions. ["AASHTO LRFD"]
2. "277609-NHB-PLN-M02-02" ["Design Drawings" or "Current Design"]
3. "277609-NHB-REP-MWER-02: US181 Harbor Bridge Replacement Project: Wind Engineering Report," Revision 2, May 4, 2021. ["Wind Report"]
4. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 1010 dated January 8, 2021 ["ISA Phase 1 Report"]
5. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 2010 dated April 23, 2022 ["ISA Phase 2 Part 1 Report"]
6. Meeting Notes of 26 May 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["May 2022 Meeting"]
7. Meeting Notes and Presentations of 10 June 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["June 2022 Meeting"]
8. Meeting Notes and Presentations of 29 July 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["July 2022 Meeting"]
9. TxDOT/HNTB Review Comments spreadsheet file: Master_Sub-4427_CRF(2021-02-11)Rev01.xlsx dated January 12, 2021 ["TxDOT/HNTB Review Comments"]
10. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Technical Memorandum TM1001 "Tower Drilled Shafts", dated August 12, 2022 ["ISA TM1001"]
11. Foundation 1ST Load Case Provided by the Developer's Lead Engineer received August 5, 2022 ["DLE 1ST Foundation Load Case"] Texas Department of Transportation (TxDOT), "Technical Provisions for US 181 Harbor Bridge Project: Comprehensive Development Agreement." ["TP"]
12. Texas Department of Transportation (TxDOT), "Technical Provisions for US 181 Harbor Bridge Project: Comprehensive Development Agreement." ["TP" or "Technical Provisions"]

3. Background

The foundation cap at 1NT and 1ST is the primary load transfer element to transmit loads from the tower legs to the drilled shafts. The geometry and reinforcement of the pile caps 1NT and 1ST are essentially the same.

For all the analysis performed by the ISA, the participation of the 24"x24" driven piles at 1NT have been neglected, per the results and findings of an independent soil structure interaction analysis performed by the ISA's Geotechnical Engineer (see Appendix B of Reference 5. above).

The tower is skewed to the foundation by a ~37.4° angle. This causes the "toes" of the tower leg to be located very close to the edge of the cap (within 2 feet). Wind loads that maximize concentrated loading at a tower leg toe cause significant shear at the edge of the cap and flexure at the face of the tower legs.

Each foundation cap is 132'-0" wide, 72'-0" long, and 18'-0" deep. Plan views showing the tower legs' footprint on the cap for 1ST (1NT similar) along with reinforcement details are presented in Figures 1 and 2 below.

AASHTO LRFD §5.7.3.2 specifies the requirements for computing flexural resistance of slabs and footings. AASHTO LRFD §5.13.3.6 specifies the requirements for computing shear resistance of slabs and footings in the vicinity of concentrated loads or reaction forces.

The appendices include applicable calculations. The calculations consider the updated (Rev. 2) Wind Report loadings.

Strength load combinations considered for the ISA of the foundation cap include the 1.05 importance factor. In the July 2022 Meeting, the Developer's Lead Engineer (DLE) argued that the 1.05 importance factor does not apply to the tower foundations. The ISA team has concluded that it is not the intent of the Technical Provisions nor sound engineering to support an essential tower with a typical foundation. Nevertheless, neglecting the 1.05 importance factor does not change the findings discussed herein.

Appendix A presents finite element modeling (FE models or FEM) of the foundation cap using the programs LARSA 4D (for thick-plate models) and Midas FEA NX (for brick-element models). The modeling focuses on the equilibrium of controlling load cases, considering wind load directions that maximizes compression at the edge of the cap from the toe of the tower leg. Reactions in each of the drilled shafts that were input into the FE models were computed using soil-structure interaction modeling, using the program, FB-MultiPier. Various load cases were considered for various wind load angles. Appendix B presents checks of applicable project requirements.

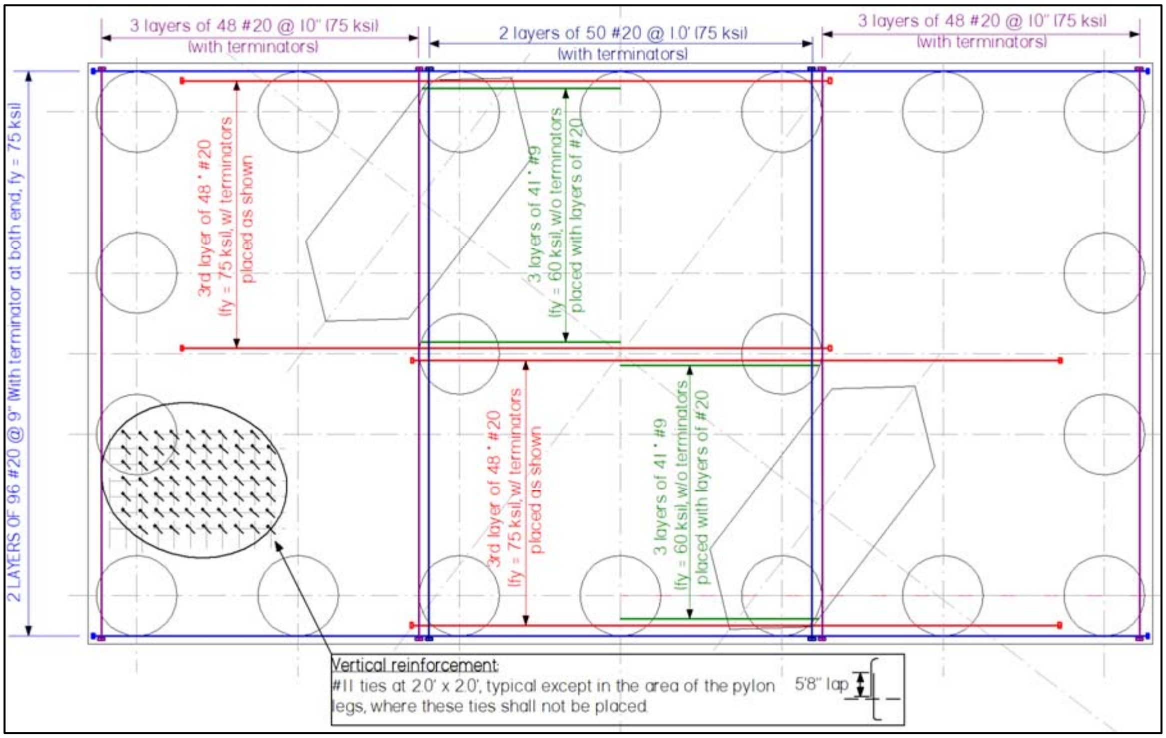


Figure 1: Foundation Cap 1ST, Bottom Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

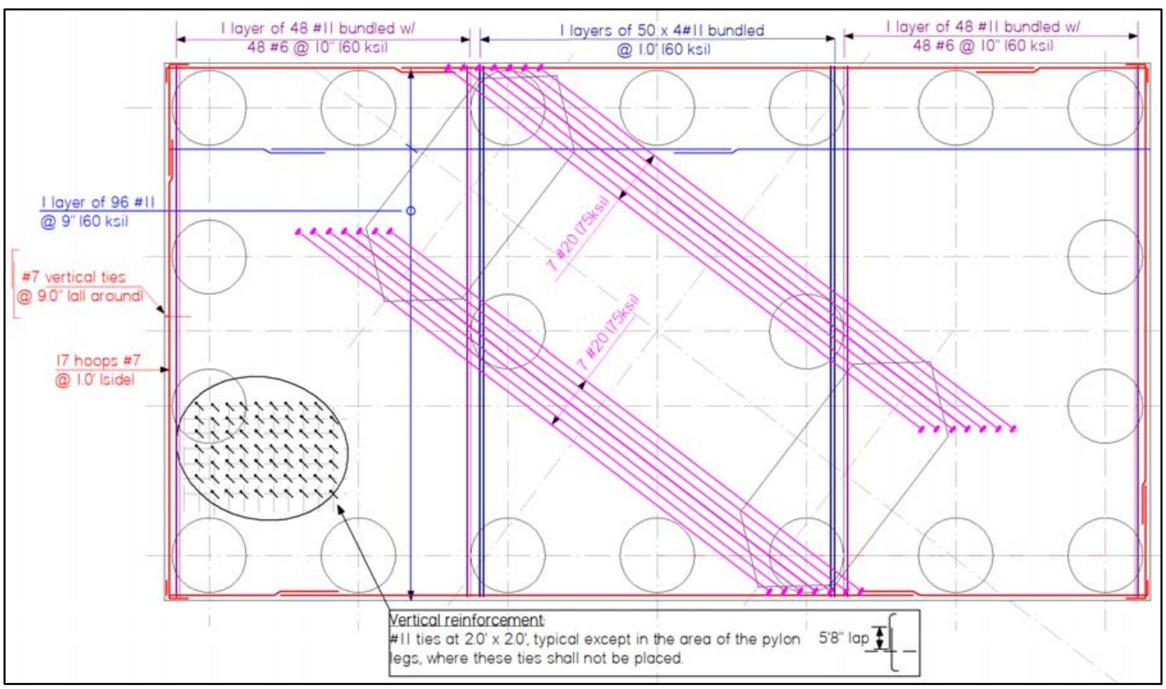


Figure 2: Foundation Cap 1ST, Top Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

4. Summary of Findings

The calculations in the appendices demonstrate that the tower foundation caps as currently designed do not meet the requirements of AASHTO LRFD §1.3.2.1, which requires that demand be less than or equal to capacity, considering the following effects:

- Transverse bending moment (causing tension in the longest bars) in the foundation cap – Resistance per AASHTO LRFD §5.7.3.2. The calculations demonstrate that with fully developed reinforcement, the demand-to-capacity ratio (D/C) exceeds the capacity of the cap on the half section closest to the leg, with a D/C = 1.25. The reinforcement to resist this transverse bending is not fully developed. Also, the location of maximum transverse bending closely coincides with sections where two-way action shear demands exceed capacity in the cap.
- One-way action in the foundation cap – Resistance per AASHTO LRFD §5.13.3.6.2 is exceeded. (see Appendix B2)
- Two-way action (punching shear) in the foundation cap – Resistance per AASHTO LRFD §5.13.3.6.3 is exceeded with D/C of 1.16 for the full perimeter, and locally up to a D/C of 4.3. (see Appendix B3 to B5)

The ISA team also applied the provisions of AASHTO LRFD §5.6.3 - Strut-and-Tie modeling in an attempt to justify the current design of the tower foundation cap subjected to this loading. However, the models considered resulted in tie demand-to-capacity ratios in excess of 2.0, and so a valid and acceptable alternate load path was not identified.

Further, the reinforcement necessary to resist transverse bending in the tower foundation cap does not meet the AASHTO LRFD §5.11.1.2 and §5.8.3.5 requirements for development and detailing of flexural reinforcement.

5. Conclusion

Flexure and shear demand exceeds capacity at the tower foundation caps subject to Strength limit state loadings. The applicable requirements are AASHTO LRFD §1.3.2.1, §5.7.3.2, §5.13.3.6.2, §5.13.3.6.3, and §5.6.3. The flexural reinforcement necessary to resist transverse bending does not meet the requirements of AASHTO LRFD §5.11.1.2 and §5.8.3.5. Unlike ductile failure, shear failure manifest itself by an abrupt loss of the ability to resist. Since there is no alternate load path, if a tower leg punches through the foundation cap, the leg would in turn lose its ability to resist thrust and bending. Thus the bridge would become unstable and collapse if cap punching shear were to occur. Insufficient development may also result in a brittle failure mode.

These findings have been presented previously in the following reports and meetings:

- ISA Phase 1 Report
- ISA Phase 2 Part 1 Report
- May 2022 Meeting
- June 2022 Meeting
- July 2022 Meeting

Some of these issues were raised in TxDOT/HNTB Review Comments (see Reference 9). For example, in Comment ID#38 TxDOT/HNTB comments that the Designer must consider punching shear of the tower legs. However, the Designer responded to that comment stating that punching shear of the tower does not govern the design. Also, the strut-and-tie method suggested in ID#20 would have revealed (as it did for the ISA team) that an adequate load path for the foundation cap does not exist for Strength limit state design loads that maximize compression at the “toes” of the tower legs, which are located too closely to the edge of the cap.

6. Response to DLE Tower Foundation Presentation

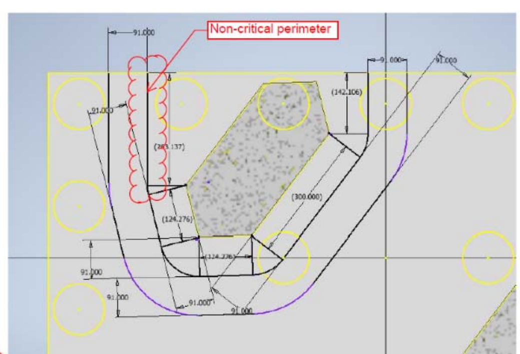
Just prior to issuing this Technical Memorandum, the ISA received design input from the DLE related to the foundation cap. While the content in other sections of this memorandum was completed prior, we provide the following technical comments in response to the DLE’s presentation:

Comments in response to Document 277609-NHB-PRESE-Tower Foundations-00, 11-Aug-2022, Pages 16-55 (foundation cap related):

1. Two-Way Shear, Pgs 27 to 30: The calculations provided by the DLE contain two significant issues in the application of AASHTO LRFD § 5.13.3.6.1 for Two-Way Shear.
    a. The critical perimeter b_o is specifically defined at a minimum distance of $0.5d_v$ from the perimeter of the reaction area. The perimeter selected by the DLE is not the critical failure plane, and the perimeter defined by the ISA is governing.
    b. The contribution of the drilled shafts shown in the DLE calculation is incorrect per AASHTO. Per AASHTO LRFD § 5.13.3.6.1, *“Where a portion of a pile lies inside the critical section, the pile load shall be considered to be uniformly distributed across the width or diameter of the pile, and the portion of the load outside the critical section shall be included in the calculation of shear on the critical section.”* The contribution from the drilled computed by the DLE shown in the figure below is significantly overestimated.

Two-way capacity (South Tower)

Shear perimeter	b0	1,240	in
Shear depth	dv	182	in
Concrete strength	f'c	5.5	ksi
Rebar yield	fy	60	ksi
Area of bar	Av	1.56	sq.in
Number of bars	n	52	
Spacing of bars	s	24	in
Column ratio	ßc	1.55	
Eq 5.13.3.6.3-1	Vn	66,668	kip
Eq 5.13.3.6.3-2	Limit	101,590	kip
Eq 5.13.3.6.3-3	Vc	33,440	kip
Eq 5.13.3.6.3-4	Vs	36,910	kip
Eq 5.13.3.6.3-2	Total	70,350	kip
Resistance factor	φ	0.9	
Cap shear resistance	Vr	63,315	kip
Contribution from piles		55,046	kip
Adjustment for cap weight		-2943	kip
Total two-way shear resistance		**115,417**	**kip**



Value significantly over-estimated

Increases to 121,000 kips based on as-built material properties and 123,000 kips with concrete age hardening.

29 ARUP CFC

Any inclusion of the drilled shaft within dv is unconservative, and the shaft contribution shall only include that portion within dv/2 per AASHTO LRFD § 5.13.3.6.1. The DLE shows approximately 2.5 shafts within this unconservative perimeter giving a pile resistance of 2.5*15,400 kips = 38,500 kips, versus the 55,046 kips presented in the table above. This results in a shear capacity of ϕVn = 63,315 + 38,500 – 2,943 = 98,872 kips. From DLE's foundations loads, Vu =111,900 kips, and therefore the **D/C is 1.13** does not meet AASHTO LRFD.

As shown in Appendix B3, and following the AASHTO LRFD 5.13.3.6.1 requirements, a more critical perimeter must be considered, that encircles the perimeter of the pier footprint by dv/2. Following this approach as defined in Appendix B3 and using the DLE's loads, the **D/C is 1.21**, which is significantly over capacity.

2. Influence of Adjacent Pile, Pgs 35 to 38: The referenced slides include consideration from the DLE to include the influence of adjacent piles. In the AASHTO LRFD requirements referenced above, the contribution of the shafts is well defined, "Two-way action, with a critical section perpendicular to the plane of the slab and located so that its perimeter, b_o, is a minimum but not closer than $0.5d_v$ to the perimeter of the concentrated load or reaction area." Clearly from this clause, the captured perimeter b_o does not extend at a 45-deg angle as proposed by the DLE.

3. Overcapacity Perimeter Leg, Pgs 31 to 34: The application of AASHTO Eq 5.13.3.6.3-2 is the root deficiency associated with punching shear of the current foundation cap design, as presented in Appendix B3.

   In this case, the AASHTO LRFD requirements are a starting point for identifying potential failure, and an understanding of the code-based behavior of the punching shear is important. It should not, however, be seen as a comprehensive summary of all potential failures. Engineers should remain alert to unanticipated potential failure modes, particularly when designing critical elements with few precedents. This is the case with the punching shear deficiency observed in the footings.

   The additional punching shear calculations presented in Appendices B4 and B5 address the irregular geometry and loading of the tower leg; the tower leg's close position to the edge of the cap; and the critical nature of punching shear as it relates to the overall stability of the bridge. All of the findings of Appendix B3, B4, and B5 are the same: demand greatly exceeds capacity for the current design of the foundation cap.

   By examining the individual legs of the shear failure surface, a serious flaw in the tower arrangement is exposed, indicating local shear demands that are over four times larger than calculated capacity, leading to the initiation of brittle failure at the free edge of the cap.

   Concern over the punching shear problem is compounded by the local transverse bending problems, where demand is at capacity, and the reinforcement has insufficient development length. These problems occur at the same localized region of the foundation cap as the two-way action (punching shear) problem identified.

LS Dyna Analysis, Pgs 40 to 55: The DLE's LS Dyna analysis appears to be a forensic study of potential post-failure behavior of the current design of the foundation cap. However, it is not consistent with AASHTO LRFD. Presumably, this analysis endeavors to demonstrate that the foundation cap will not collapse as a result of the factored loads, and it shows behaviors consistent with plastic design of the foundation. Without commenting on the accuracy of this analysis, those behaviors and conclusions would only be appropriate for an Extreme limit state case, where collapse prevention is the minimum requirement. They are not consistent with a Strength limit state load case, where the minimum requirement is meeting a target reliability through consistency with the AASHTO LRFD requirements.

Appendix A. FE model presentation

1. <u>Appendix A1 – Plate Element Finite Element Model for Evaluation of Bending Demand and One-Way Action</u>

A plate element model has been created in LARSA 4D to evaluate the demand in the pile cap at the critical sections defined in AASHTO LRFD for the verification of bending and shear capacity, as shown in Figure 3 below. For each foundation load case considered, concomitant demand in the North and South tower legs are applied to the pile cap plate model. The resulting reactions in each of the drilled shafts have been evaluated using the FB-Multipier and applied to the pile cap plate model, resulting in the same pile cap equilibrium as that assumed for the ISA evaluations of the drilled shafts (refer to the ISA technical memorandum TM1001 for more details).

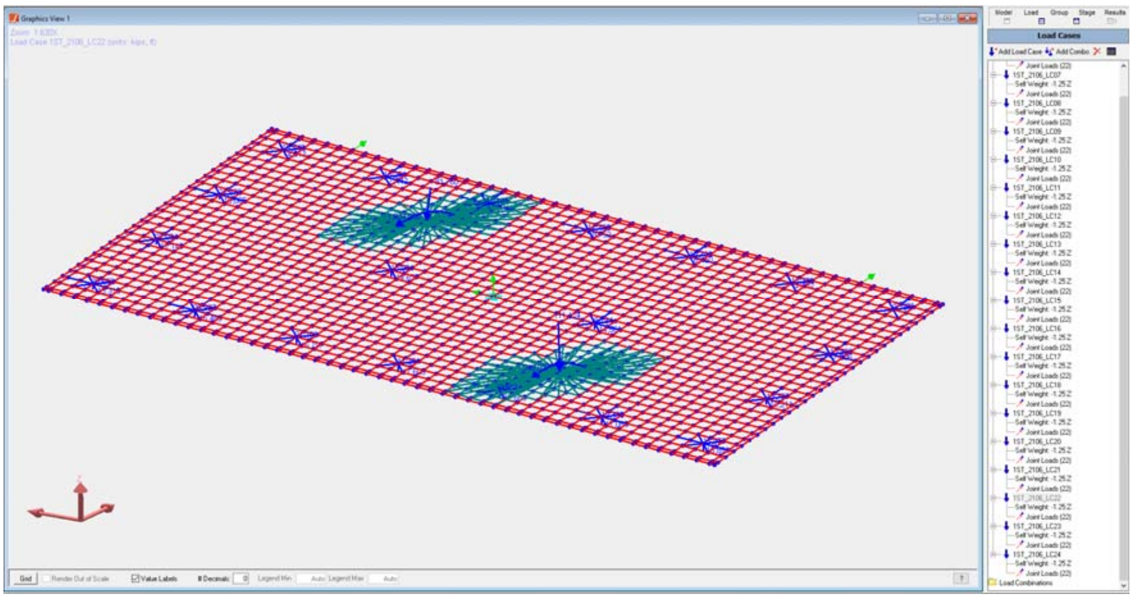


Figure 3: FEM Model of Tower Foundation Cap 1ST (1NT similar)

Per AASHTO LRFD § 5.13.3.6.1 and § 5.8.3.2, the critical section for one-way action extends across the entire width of the element and is located at a distance d_v from the face of the pier, with d_v being the effective shear depth of the section, which is d_v = 187 in for this foundation cap. The critical section for one-way action is shown as Σ_1' in the Figure 4 below.

Per AASHTO LRFD § 5.13.3.6.1, the critical section for two-way action around each pylon leg is perpendicular to the plane of the slab and located so that its perimeter, b_0, is a minimum but not closer than $0.5d_v$ to the perimeter of the pylon leg, as shown in the Figure 4 below.

Per AASHTO LRFD § 5.13.3.4, the critical sections for bending in footings is taken at the face of the pier. The critical section for bending is shown as Σ_1 and Σ_2 in the Figure 4 below.

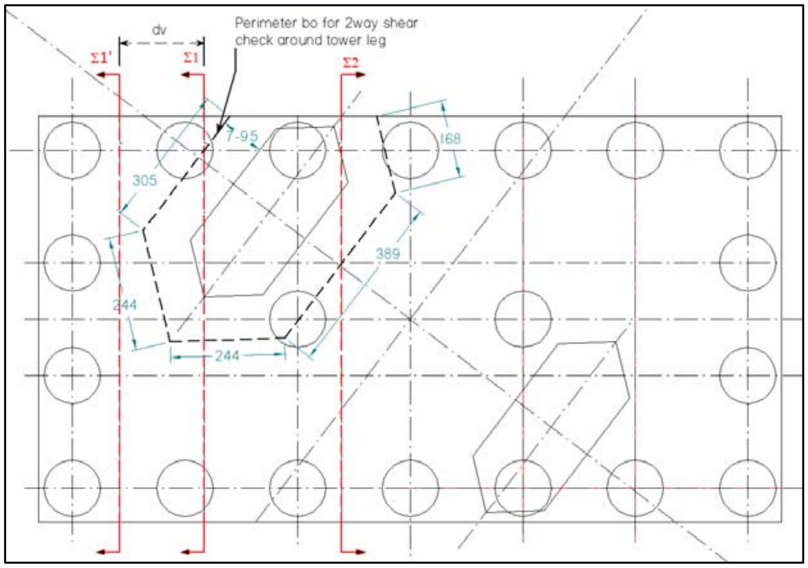


Figure 4: Critical Section Definition for Bending (Σ_2), One-Way Action (Σ_1') and Two-Way Action (Perimeter b_o)

2. Appendix A2 – Brick Element Finite Element Model for Evaluation of Local Shear Stress Distribution Around the Tower Legs for Two-Way Action

The local shear stress distribution over the critical perimeter b_o for the checking of the two-way action around the tower legs has been evaluated with a brick-element finite element model generated using Midas FEA NX finite element software. Like the plate element model, the loads applied to the model have been generated to recreate the total pile cap equilibrium under the governing Strength load combination, as determined by FB-MultiPier soil-structure interaction modeling software.

An overview of the brick element model is provided in Figure 5 below.

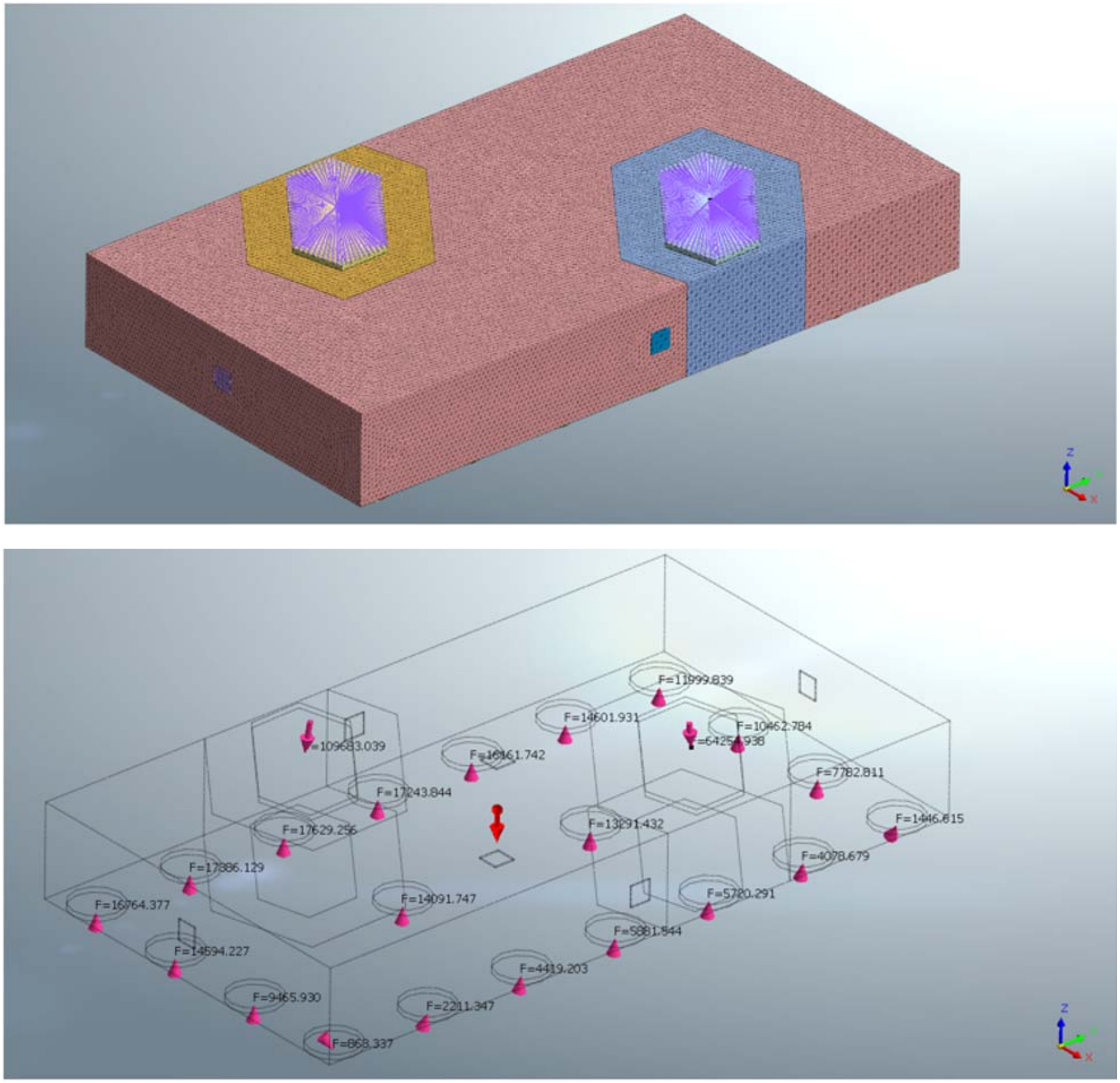


Figure 5: MIDAS FEA NX Brick Element Model - Geometry

Appendix B. Analysis Results

1. Appendix B1 – Transverse Bending

The critical section for the transverse bending check is shown in Figure 6 below, representing the transverse bending moment distribution on the pile cap for the governing Strength III load combination (maximum wind). The tower leg demands corresponding to this load case are provided in the table below.

Load combination	N-W leg, node 125						S-E leg, node 61 (loads concomitant with loads on 125)					
	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips
STR 3 with 1.05 Importance factor	-11,678	-9,801	-110,073	-561,794	-749,025	83,668	-578	10,780	-81,044	-614,999	-824,469	-106,391

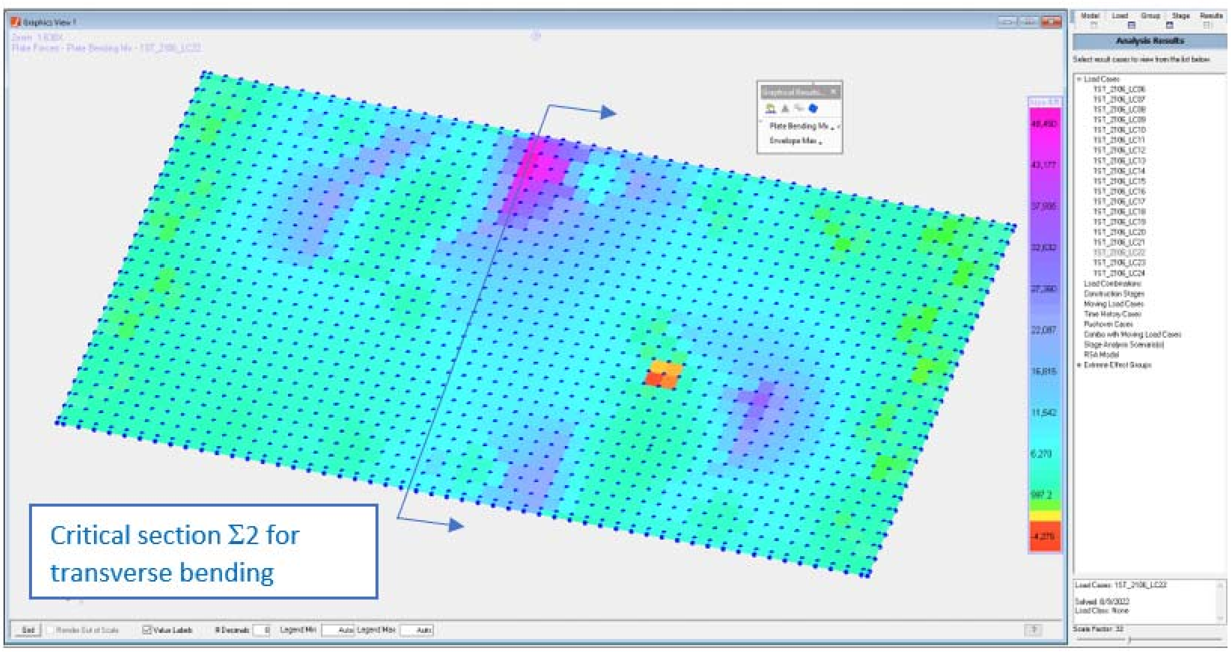
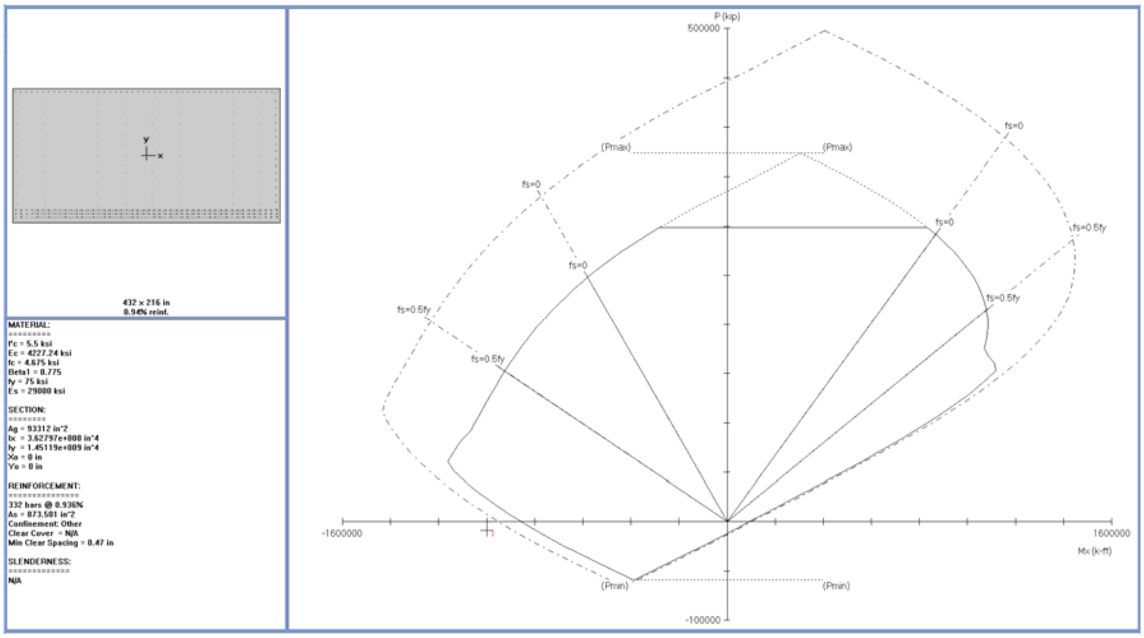


Figure 6: Transverse Bending Demand under Governing Strength III Load Combination (η_I = 1.05)

As shown in the Figure 6, the bending demand is concentrated on the side of the section closest to the pier leg carrying the maximum axial load. An attempt to address this concentration of the bending demand has been considered in the design, as evidenced by the local increase of bending reinforcement provided in this area: refer to Figure 1, where 3 layers of 41 #9 bars [bar marks F0901 on drawing NHB 55] are provided at this ½ section. Although the axial + bending demand capacity ratio over the full section is adequate, the ISA team estimates that given the dimension of the foundation cap and the localized nature of the demand, the local reinforcement must be sized to adequately resist the localized demand over a ½ section. With the reinforcement provided, the bending demand exceeds the capacity with a demand-to-capacity ratio, D/C = 1.25, as shown in the interaction plot in Figure 7. Also, bars F0901 should be extended by an additional 5' past section Σ2 to be fully developed, per AASHTO LRFD §5.11.2 and §5.8.3.5. (see Figure 8).



Factored Loads and Moments with Corresponding Capacity Ratios

NOTE: Calculations are based on "Moment Capacity" Method.
Section capacity exceeded. Revise design!

No.	Demand		Capacity		Parameters at Capacity			Capacity
	Pu	Mux	ϕPn	ϕMnx	NA Depth	εt	ϕ	Ratio
	kip	k-ft	kip	k-ft	in			
1	-8539.00	-998864.00	-8539.00	-797882.13	30.13	0.01761	0.900	1.25

Figure 7: Interaction Diagram for Governing Strength III Demand on 1/2 Section S2: Demand / Capacity = 1.25

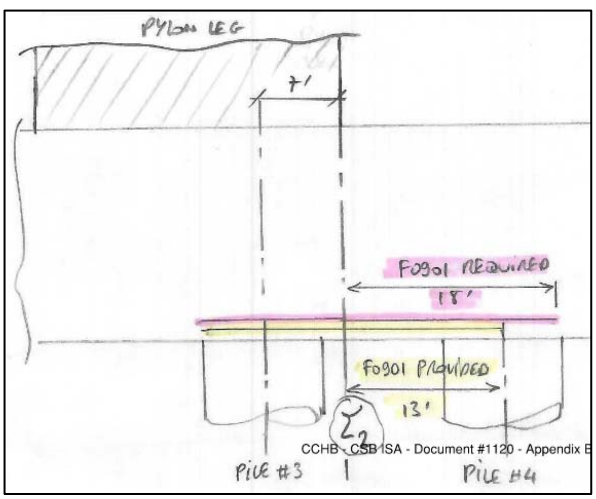


Figure 8: Insufficient Development of Bar F0901 Past Section Σ2, per AASHTO LRFD §5.8.3.5

2. Appendix B2 – One-Way Action

For one-way action, the depth d_v = 187 in, based on the geometric characteristics of the section and the distribution of reinforcement.

Per LRFD § 5.8.3.4.1 $\beta = 2.0$; $\theta = 45^\circ$.

Demand has been evaluated at the Section Σ_1' (defined in Figure 4Figure 4) for the governing Strength III load combination provided in the table below:

Load combination	N-W leg, node 125						S-E leg, node 61 (loads concomitant with loads on 125)					
	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips
STR 3 with 1.05 Importance factor	-4,298	-1,270	-55,554	-625,069	-786,219	-4,336	6,569	13,785	-112,465	-671,470	-74,269	-112,524

The one-way shear capacity of the critical section has been evaluated per the recommendation of AASHTO LRFD §5.8.3.3, assuming ϕ = 0.9 per LRFD §5.5.4:

$V_r = \phi V_n$ with $V_n = \text{Min}(V_c + V_s\,;\ 0.25\, f'_c\, b_v\, d_v)$.

For all transverse sections, the shear capacity V_r is evaluated as follows: d_v = 187"; b_v = 72' = 864"

$V_{r\,max} = \phi\, V_{n\,max} = \phi \times 0.25 \times f'_c \times b_v \times d_v$ = 199,940 kips ; $V_c = 0.0316\, \beta\, \text{sqrt}(f'c)\, b_v\, d_v$ = 23,947 kips

$Vs = \frac{A_v f_y d_v}{s} . \cot(\theta)$, with $\cot(\theta)$ = 1.0

A_v/s provided = # 11 @ 2' x 2'. Assuming a width of the transverse section of 72', 36 sets of #11 ties (F1111 and F1106) are assumed to be provided per set → A_v/s provided = (36 * 1.56 in^2) / 2 = 28.1 in^2 / ft = 2.34 in^2 / in.

#7 bars are provided on each side at 10" spacing, adding A_v/s provided' = (2*0.6 in^2)/10 in = 0.12 in^2/in.

Total A_v/s provided = 2.34 in^2 + 0.12 in^2 = 2.46 in^2/in.

Vs = 2.46 in^2/in * 60 ksi * 187 in = 27,601 kips → $V_r = \phi\,(V_c + V_s)$ = 0.9*(23,947+27,601) = 46,393 kips

One-way shear capacity for the entire transverse section: V_r = 46,393 kips

Governing Load case: STR III, with η_I = 1.05	Vu (kips)	Vr (kips)	D/C
1/2 section transverse Σ1' North	15,366	23,197	0.66
1/2 section transverse Σ1' South	31,739	23,197	**1.37**
Total transverse section Σ1'	47,105	46,393	**1.015**

Demand exceeds capacity for the entire section Σ_1' with a D/C ratio of 1.015.

Like the bending demand, the shear demand is highly concentrated on the half section located closest to the tower leg. Although AASHTO LRFD §5.13.3.6.1 states that the critical section extends across the entire width of the footing cap, it is appropriate to evaluate the demand and capacity considering only the half width of the section, especially given the unconventional positioning of the pylon legs on the cap. Demand exceeds capacity for the half section Σ_1' with D/C = 1.37.

3. Appendix B3 – Two-Way Action

The two-way action checks around the pier leg have been carried out by the ISA team following the recommendation of AASHTO LRFD §5.13.3.6.1. The critical perimeter b_o is defined in Appendix A1 above. As illustrated in Figure 9 below, portions of drilled shafts 2, 3, 4, and 19 fall inside the perimeter b_o. Per AASHTO LRFD §5.13.3.6.1, *"Where a portion of a pile lies inside the critical section, the pile load shall be considered to be uniformly distributed across the width or diameter of the pile, and the portion of the load outside the critical section shall be included in the calculation of shear on the critical section."*

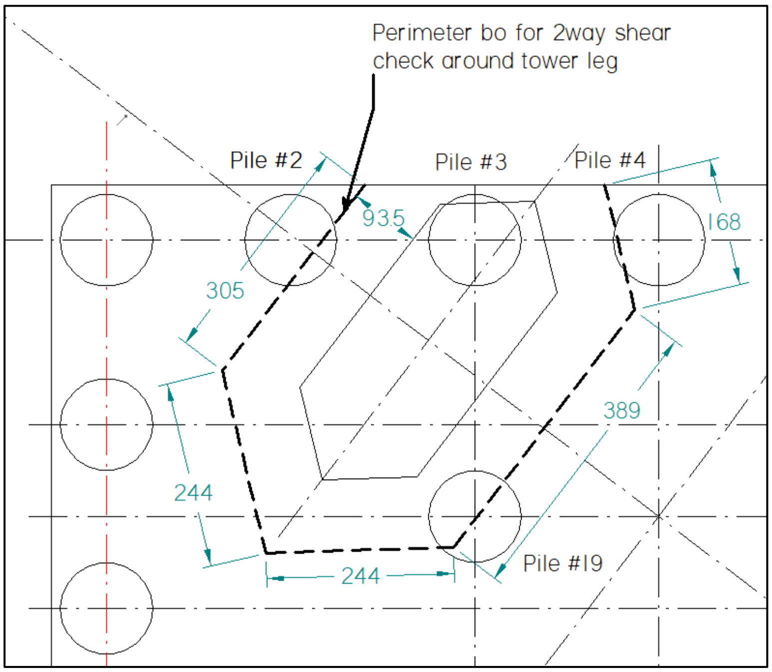


Figure 9: Detail of Critical Perimeter b_o for 2-Way Action: Dimension, Distance to Tower Leg, Intersection with Drilled Shafts.

The total shear Vu is calculated for the governing Strength III load combination, based on the percentage of each pile area located within the perimeter: Vu = 81,617 kips

Load combination	N-W leg, node 125						S-E leg, node 61 (loads concomitant with loads on 125)					
	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips	Fx kips	Fy kips	Fz kips	Mx kips	My kips	Mz kips
STR 3 with 1.05 Importance factor	-11,678	-9,801	-110,073	37,898	-749,025	83,668	-578	6,154	-63,083	-614,999	-824,469	-6,208

Check:	Pu (kips)	% inside bo	total inside bo (kips)
Pu on leg	110,073	100.00%	110,073
Pu pile 3	-17,649	100.00%	-17,649
Pu pile 19	-14,160	51.70%	-7,321
Pu pile 2	-17,424	17.30%	-3,014
Pu pile 4	-17,240	2.74%	-472
total Vu applied on b_o =			81,617

The two-way shear capacity has been evaluated based on AASHTO LRFD §5.13.3.6.3. for a section with transverse reinforcement.

Perimeter b_o = 305 + 244 + 244 + 389 + 168 = 1,350 in

Effective shear depth d_v = 187 in.

$$V_n = V_c + V_s \le 0.192\sqrt{f'_c}\; b_o d_v \qquad (5.13.3.6.3\text{-}2)$$

Shear capacity from concrete: $V_c = 0.0632 \text{ sqrt } (f'_c)\; b_o\, d_v$= 37,417 kips (AASHTO LRFD Eq. 5.13.3.6.3-3)

Shear capacity from reinforcement: $V_s = (A_v/s).\ f_y\ .\ d_v$ (AASHTO LRFD Eq. 5.13.3.6.3-4)

A_v/s provided = # 11 @ 2' x 2'

→ A_v/s provided per unit length of perimeter= 1.56 in^2/ (2*12 in)/(2*12 in) = 0.002708 in^2/in/in

A_v/s provided over the perimeter bo = 0.002708 in^2/in/in * 1,350 in = 3.656 in^2/in

$V_s = (A_v/s).\ f_y\ .\ d_v$ = 3.656 in^2/in * 60 ksi * 187 in = 41023.1 kips

$V_r = \phi\,(V_c + V_s)$ = 0.9*(37,417 + 41,023) = 70,596 kips < 0.9 * 0.192 sqrt $(f'_c)\; b_o\, d_v$ = *102,305 kips*

V_u = 81,617 kips > V_r = 70,596 kips → D/C = V_u / V_r = 1.16

Using the demand on the tower leg evaluated by the ISA at the Strength limit state, the two-way shear demand around the tower leg exceeds the two-way shear capacity of the foundation cap with D/C = 1.16.

Note: The foundation loads provided by the Developer's Lead Engineer (DLE) show the maximum axial load on the governing tower leg on the foundation cap 1ST as Pu = 111,900 kips (see DLE 1ST Foundation Load Case). Based on the design methodology stated by the DLE during the June 2022 Meeting, the DLE assumes that the drilled shaft behavior become fully plastic once the axial demand reaches the geotechnical axial capacity Pr = 15,400 kips (Foundation 1ST). Using the DLE loads, the two-way shear demand on the perimeter b_o may be computed as Vu = 85,452 kips (see table below) , which exceed the capacity of the foundation cap in two-way shear with D/C = 1.21. (This observation is not intended as concurrence with the analytical approach taken by the DLE, only to note the results for comparison.)

DLE loading - STR III - Foundation Cap 1ST			
Check:	**Pu (kips)**	**% inside bo**	**total inside bo (kips)**
Pu on leg	111,900	100.00%	111,900
Pu pile 3	-15,400	100.00%	-15,400
Pu pile 19	-15,400	51.70%	-7,962
Pu pile 2	-15,400	17.30%	-2,664
Pu pile 4	-15,400	2.74%	-422
total Vu applied on b_o =			**85,452**

4. Appendix B4 – Two-Way Action: Shear Stress Evaluation

As for the transverse bending and the one-way action, the unusual configuration of the tower foundation caps results in the presence of a large shear load along the edge of the pile cap. This has been evaluated using a brick element finite element model described in Appendix A2 of this memorandum with results presented in Figures 10, 11, and 12 below.

The average shear stress demand under Strength III in the area AB shown in Figure 10 varies between v_u = 1.2 ksi at node A and v_u = 0.4 ksi at node B, which can be compared to the shear stress capacity evaluated in Appendix B3 above for the perimeter bo: *vr =* $V_r / (b_o.d_v)$ *= 70,596 kip / (1,350 in * 187 in) = 0.28 ksi.*

Over the face AB, the shear stress exceeds the capacity with D/C ranging between 4.3 and 1.4.

Similarly, the average factored shear stress demand under Strength III in the area EF shown in Figure 12 varies between v_u = 0.28 ksi at node E and v_u = 0.48 ksi at node F.

Over the face EF, the shear stress exceeds the capacity with D/C ranging between 1.0 and 1.7.

The finding above is based on a stress-based evaluation of two-way action. It is important to point out that this computation is consistent with ACI-318-14, a cited reference in AASHTO LRFD Chapter 5. In the 2014 Edition, ACI-318 switched from force-based to stress-based calculations of two-way action. This change in approach prevents an “unzipping” type failure due to two-way action (punching shear), where one side or corner of the failure perimeter is highly loaded, and other sides or corners are not as highly loaded. This is the case for the tower foundation caps, subject to the design loadings considered herein. AASHTO LRFD has not switched from force-based to stress-base, and this tends to greatly understates the D/C ratio for this particular case. However, the AASHTO LRFD force-based calculation still gives demand significantly exceeding capacity for the foundation cap, as demonstrated in Appendix B3 above.

Note: The Developer’s Lead Engineer (DLE) have provided their governing foundation load case for the foundation cap 1ST (see DLE 1ST Foundation Load Case). For comparison purposes, this set of loads has been applied in the ISA FB-MultiPier finite element model of the foundation to determine the reaction on each of the drilled shafts following the same methodology used by the ISA team to determine the force equilibrium on the foundation cap. The equilibrium obtained using the DLE loads has then been applied to the ISA brick finite element model (in MIDAS FEA NX) to evaluate the shear stress along the critical perimeter b_o. The results of this analysis are presented in results presented in Figure 13 and Figure 14 below.

Using the DLE foundation loads, the average shear stress over the face AB varies between 1.0 ksi and 0.4 ksi, which exceeds the capacity with D/C ranging between 3.5 and 1.4.

Using the DLE foundation loads, the average shear stress over the face EF varies between 0.27 ksi and 0.43 ksi, which exceeds the capacity with an average D/C ratio of 1.33.

Appendix B5 – Maximum Two-Way Shear Limit

The maximum factored shear (averaged through the depth of the section) from the brick-element finite element model shown in Figure 10 is 1.2 ksi. This agrees with results from plate element models described herein and with the models discussed in the ISA Phase 1 Report and the ISA Phase 2 Part 1 Report. Regardless of the amount of reinforcement provided, both AASHTO LRFD and ACI-318 limit the maximum factored allowable shear. AASHTO LRFD and ACI limits shear to $\phi\ 0.192 \sqrt{f'c}$. With f'c = 5.5 ksi, this results in an allowable shear stress of 0.40 ksi compared to the demand of 1.2 ksi noted above.

Therefore, even if the maximum amount of shear reinforcement was provided (and it was not), the foundation cap section, as currently designed, cannot resist the design loading considered. The foundation cap needs to be thicker, or the tower legs need to be located further away from the edges, or the tower loads need to decrease, or some combination thereof.

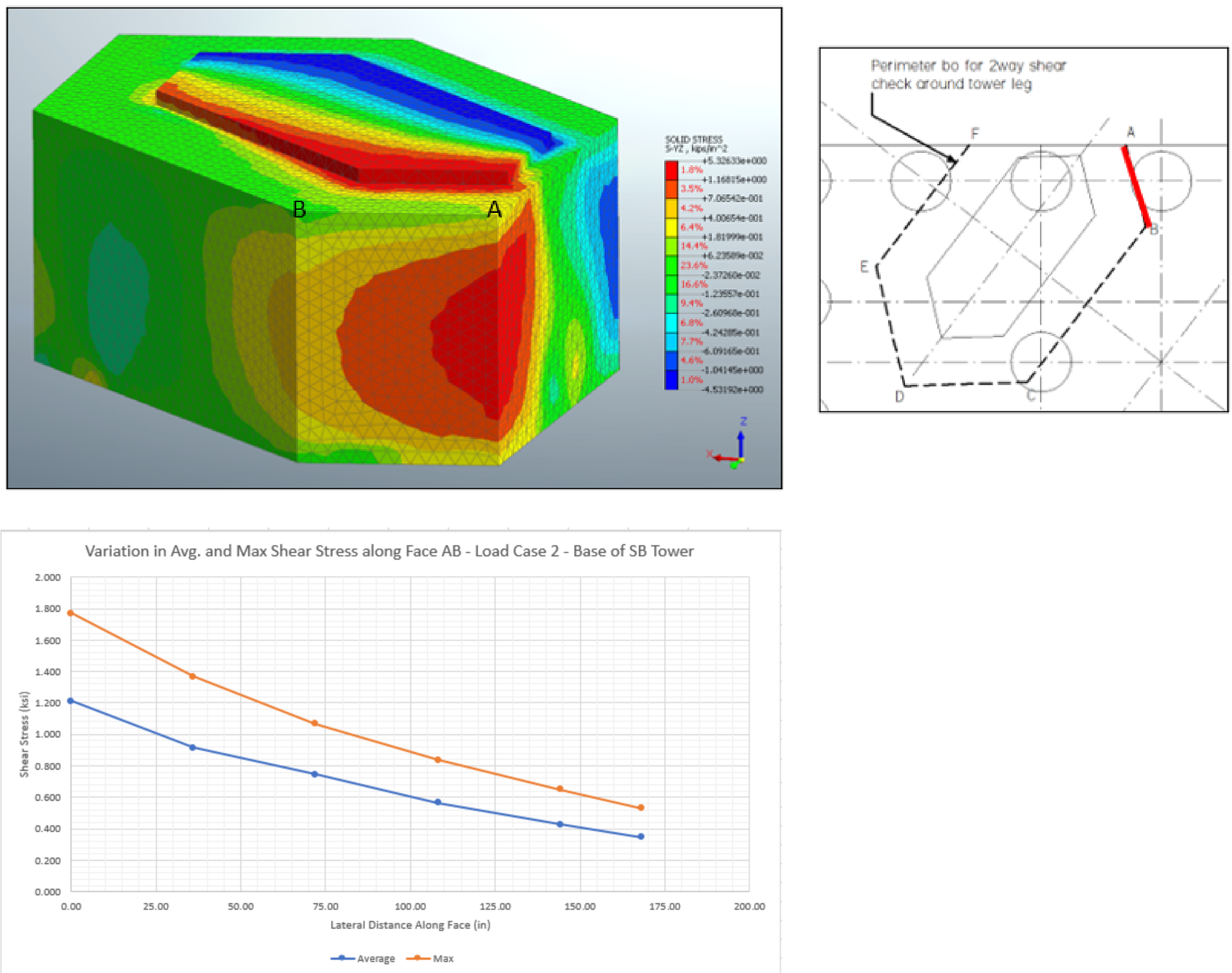


Figure 10: Shear Stress Distribution Along Plane AB for ISA Governing STR III Load Combination

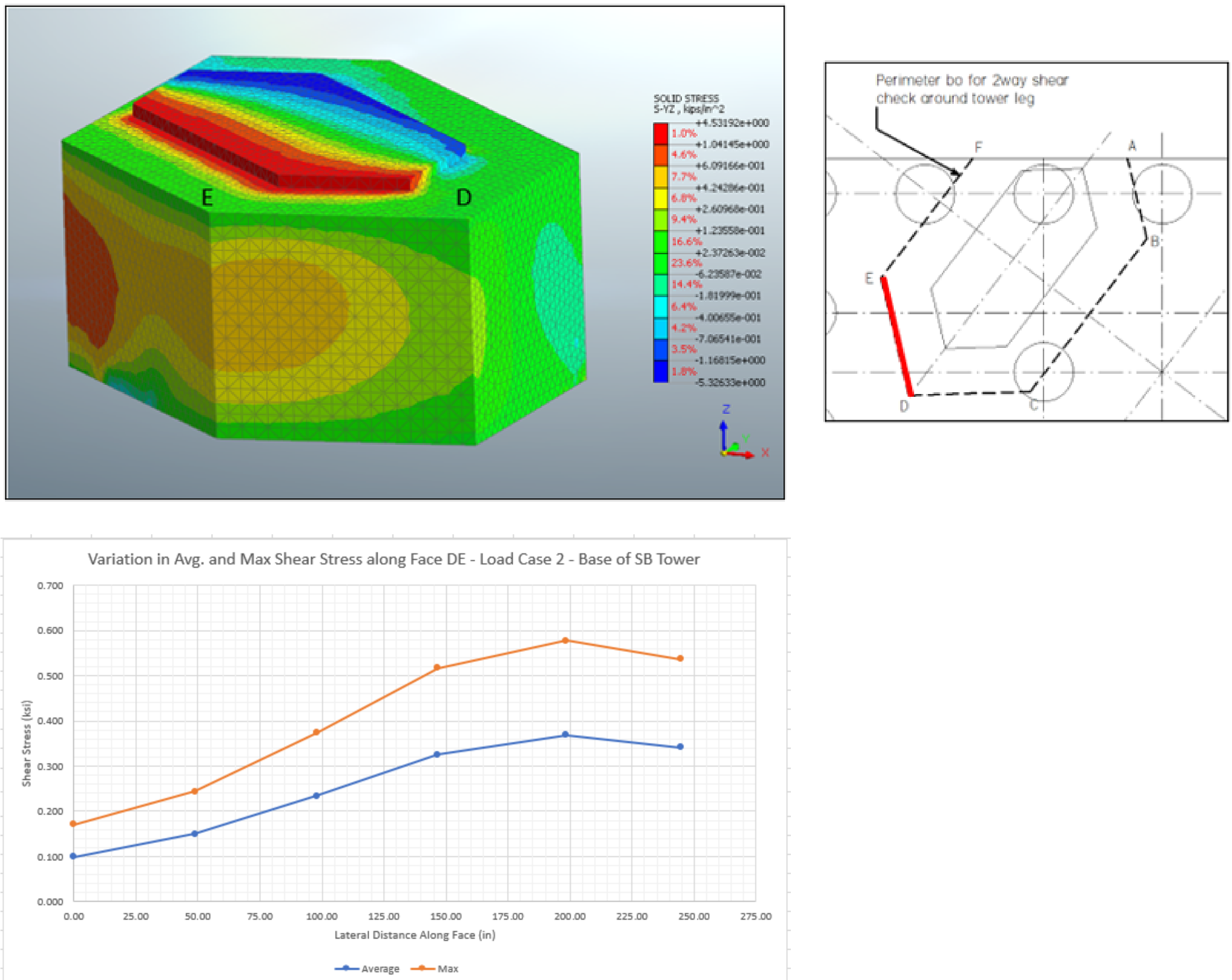
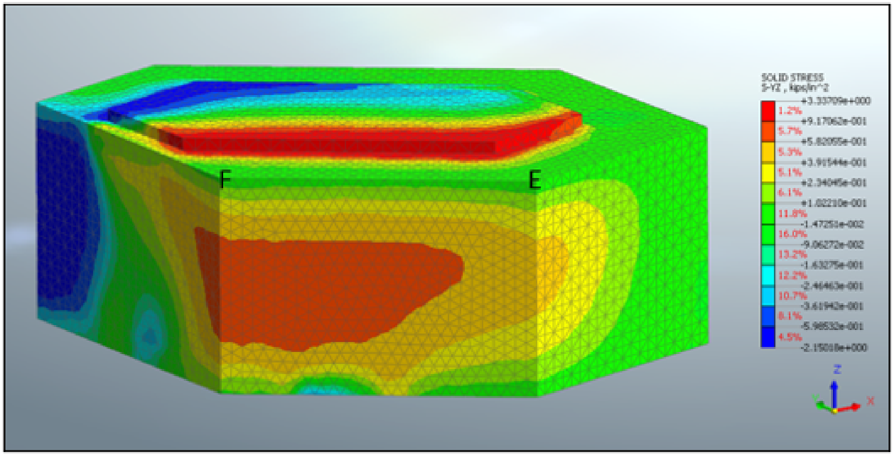
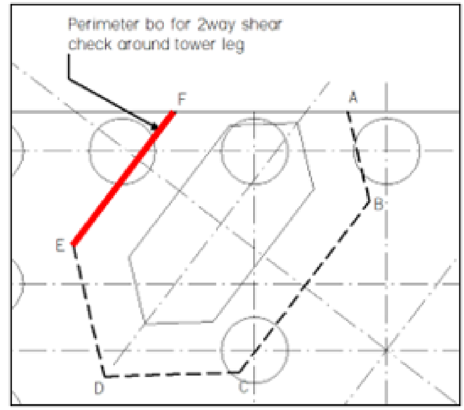


Figure 11: Shear Stress Distribution Along Plane DE for ISA Governing STR III Load Combination





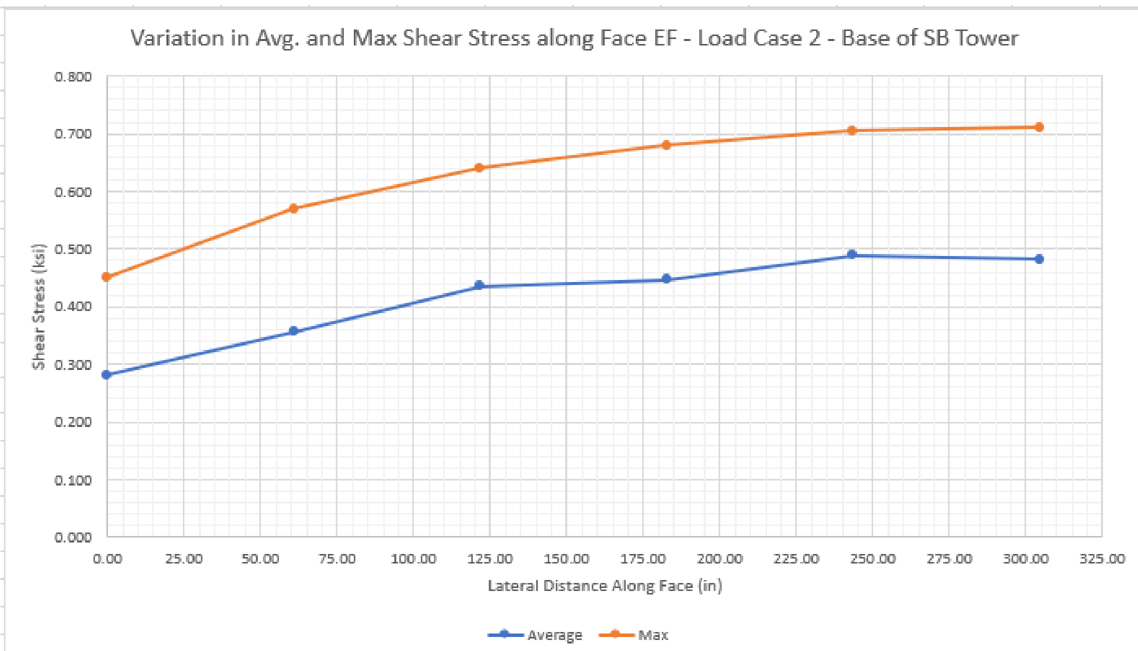
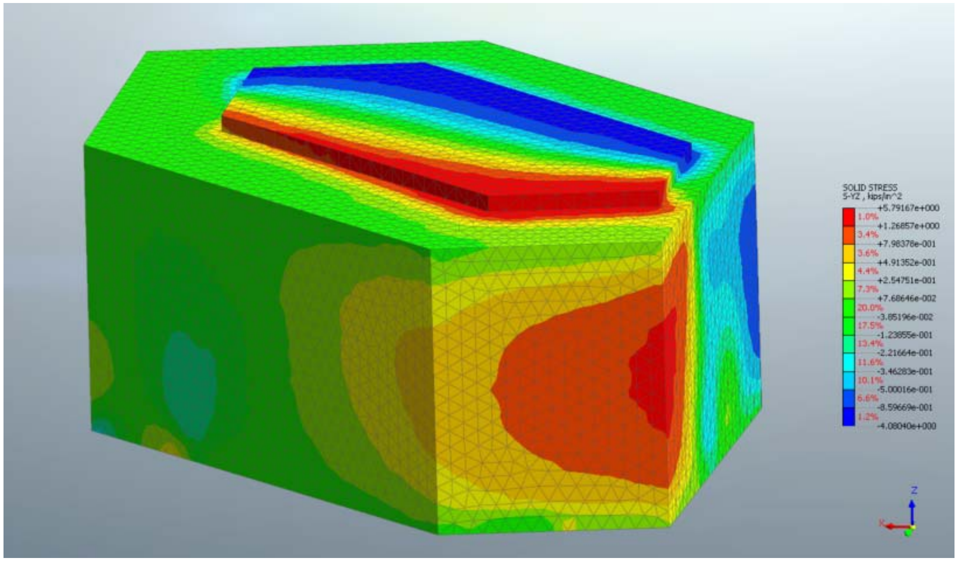
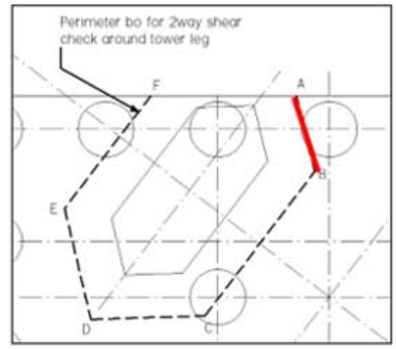


Figure 12: Shear Stress Distribution Along Plane EF of Perimeter bo for ISA Governing STR III Load Combination





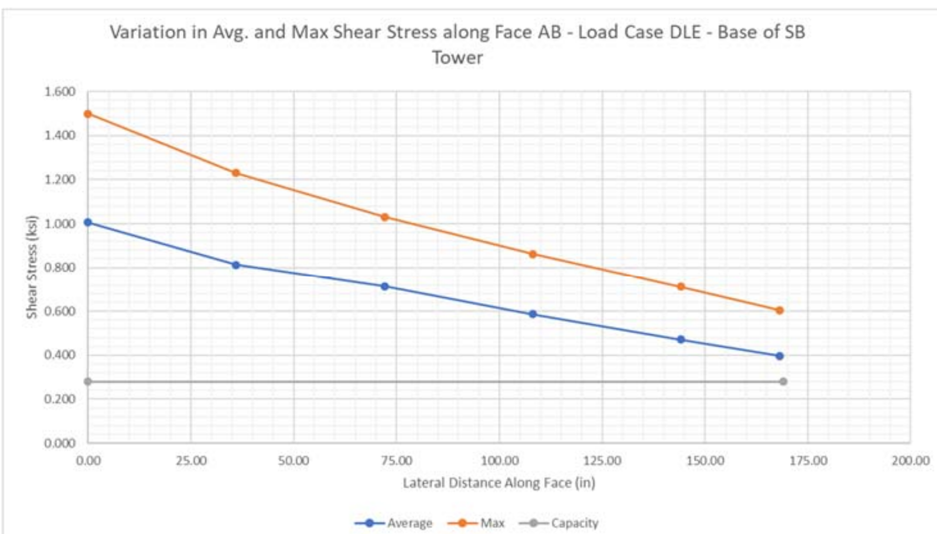
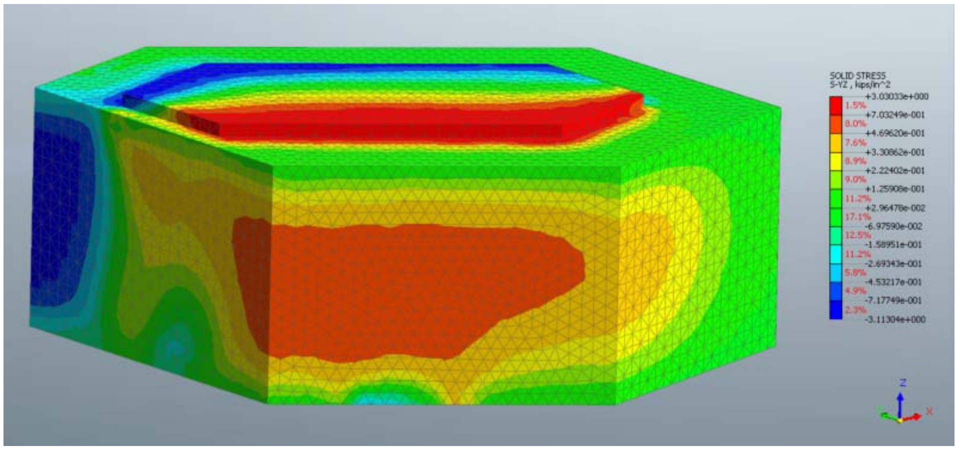
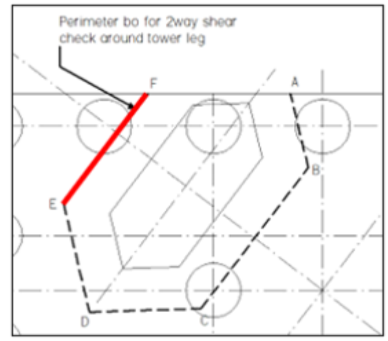


Figure 13: Shear Stress Distribution Along Plane AB of Perimeter b_o Evaluated by the ISA Team Based on DLE Foundation Loads





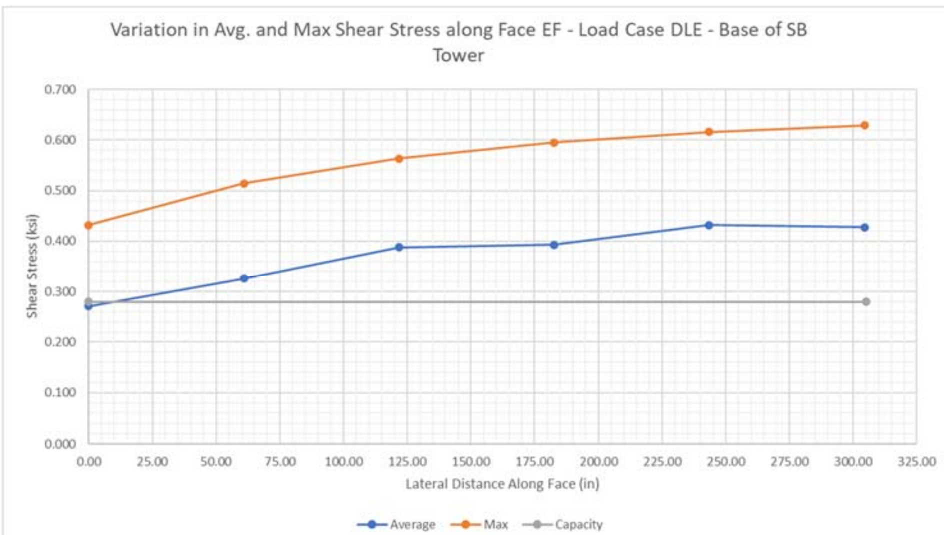


Figure 14: Shear Stress Distribution Along Plane EF of Perimeter b_o Evaluated by the ISA Team Based on DLE Foundation Loads