Texas Department of Transportation

125 EAST 11TH STREET, AUSTIN, TEXAS 78701-2483 | 512.463.8588 | WWW.TXDOT.GOV

August 16, 2022

Mr. Keith Armstrong
Project Manager
Flatiron/Dragados, LLC
1620 N Port Ave
Corpus Christi, Texas 78401

RE: US 181 Harbor Bridge Replacement Project
CSJ# 0101-06-095
Notice of Developer Default

Dear Mr. Armstrong:

The Texas Department of Transportation ("TxDOT") hereby provides Flatiron/Dragados, LLC ("FDLLC") with notice that a Developer Default has occurred pursuant to Section 16.1.1(c) of the Comprehensive Development Agreement ("CDA") between the TxDOT and FDLLC. TxDOT issues this notice ("Notice of Developer Default") in accordance with Section 16.1.2 of the CDA. Capitalized terms not defined in this notice shall have the meanings given to them in the CDA.

On April 29, 2022, TxDOT provided to FDLLC a Notice of Nonconforming Work identifying a number of separate instances involving failure of the design and construction of the New Harbor Bridge to comply with the CDA. Along with the Notice of Nonconforming Work, TxDOT forwarded a copy of a signed and sealed report from Systra International Bridge Technologies ("IBT") dated April 23, 2022, which detailed those concerns. The IBT report contained IBT's findings based on an independent design review it conducted with respect to the design for the New Harbor Bridge prepared by FDLLC's Engineer of Record, Arup-CFC. As TxDOT explained in the Notice of Nonconforming Work, the design and construction deficiencies described in the IBT report are serious and therefore demanded immediate attention:

> *The IBT Report identifies a number of instances of Nonconforming Work that raise significant and alarming concerns with the design and construction of the New Harbor Bridge. The findings in the IBT Report establish that FDLLC has failed to provide a design for the New Harbor Bridge that ensures the safety and integrity of the New Harbor Bridge. In short, FDLLC has failed in many significant and material ways to perform the Work on the New Harbor Bridge in accordance with the Contract Documents.*

As prescribed by Section 6.8.1 of the CDA, FDLLC was required, within ten days of its receipt of the Notice of Nonconforming Work, to remove and replace the Nonconforming Work so as to conform to the requirements of the CDA. If the Nonconforming Work could not be corrected within that period, then FDLLC was required, within ten days, to (a) provide to TxDOT a schedule acceptable to TxDOT for correcting all such Nonconforming Work, (b) commence such corrective work within such ten-day period, and (c) thereafter diligently prosecute such correction to completion in accordance with the approved schedule.

Since that time, at FDLLC's request, TxDOT, FDLLC, IBT and Arup-CFC have engaged in multiple meetings to allow FDLLC and Arup-CFC to ask questions and exchange information regarding IBT's findings. TxDOT asked IBT to review and incorporate the information from those meetings and to expedite a supplemental analysis focused on the following five items of Nonconforming Work that TxDOT has determined present the greatest safety issues:

1) inadequate capacity of the pylon drilled shafts,
2) deficiencies in footing caps that led IBT to report that the bridge would collapse under certain load conditions,
3) delta frame design defects, primarily related to the connections between the delta frames and the adjacent precast box units,
4) significant uplift at the intermediate piers, and
5) excessive torsion and other stresses during construction.

On August 12, 2022, IBT provided to TxDOT signed and sealed updates to its April 23, 2022 report that considered the additional information provided in the meetings with FDLLC and Arup-CFC. Copies of these updates are attached to this notice. These updates demonstrate that the fundamental findings of IBT with respect to the five most critical items remain unchanged and reconfirm the conclusion that those five items constitute Nonconforming Work.

In the three and one-half months since TxDOT delivered the Notice of Nonconforming Work, FDLLC has failed to take appropriate action to correct the issues raised in the Notice of Nonconforming Work,[1] including failing to provide a schedule for correcting any of critical items or other elements of Nonconforming Work, even though TxDOT has repeatedly requested such a schedule. Instead, FDLLC has repeatedly asserted that there is no Nonconforming Work, and that every aspect of the design and construction of the New Harbor Bridge conforms completely to the requirements of the CDA. In other words, FDLLC has made it clear that it completely disagrees with and disputes IBT's findings, and that FDLLC will not take corrective action.

Attached hereto as Exhibit A is a list of the items that constitute Nonconforming Work, which were all raised in the April 29, 2022 Notice of Nonconforming Work. As set forth in Section 16.1.1(c) of the CDA, the failure of FDLLC to perform the Work in accordance with the CDA, including failure to conform to applicable standards set forth therein in design and construction of the Project, constitutes a Developer Default. FDLLC's refusal to correct, remove and replace Nonconforming Work after notice from TxDOT also constitutes a Developer Default. Although these Developer Defaults have previously been brought to FDLLC's attention, as set forth above, this letter serves as TxDOT's formal notice of these Developer Defaults in accordance with Section 16.1.2 of the CDA. Pursuant to Section 16.1.2 of the CDA, FDLLC has a fifteen-day cure period with respect to these Developer Defaults, which begins on the date of FDLLC's receipt of this Notice of Developer Default. If Developer fails to cure the Developer Defaults identified in this notice within that fifteen-day cure period, TxDOT has the right, at any time after expiration of the cure period, to declare that an Event of Default has occurred.

[1] IBT determined that, based on information learned in the meetings, Item No. 15 from the Notice of Nonconforming Work is no longer an issue. That item is not included in the attached list of deficiencies.

Sincerely,

DocuSigned by:

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Brian R. Barth, P.E.
Deputy Executive Director-Program Delivery
Texas Department of Transportation

Attachments: Exhibit A: Summary of Selected IBT Findings and CDA Provisions
Exhibit B: TM1001_Tower Drilled Shafts_2022-08-12
Exhibit C: TM1002_Tower Foundation Cap_2022-08-12
Exhibit D: TM1003_Delta Frame-to-Box Girder Connection_2022-08-12
Exhibit E: TM1004_EJ Pier Segment-Uplift_2022-08-12
Exhibit F: TM1005_Erection_Loading_2022-08-12

cc: Kurt Knebel, Flatiron Constructors, Inc.
Justo Molina, Flatiron/Dragados, LLC
Jose Antonio Lopez-Monis Plaza, Dragados USA
Javier Sevilla, Flatiron Constructors, Inc.
John Couture, Vice President, Flatiron Constructors, Inc.
Jamie Hurtado Cola, Legal Counsel, Dragados, SA
Kyle Bogdan, Legal Counsel, Flatiron Constructors, Inc.
Marc D. Williams, P.E., Executive Director, TxDOT
Valente Olivarez, Jr., P.E., TxDOT

Liberty Mutual Insurance Company
Attn: Sam Barker
Surety Claims Counsel
P.O. Box 34526
Seattle, WA 98124-1526

EXHIBIT A
SUMMARY OF SELECTED IBT FINDINGS AND CDA PROVISIONS2

IBT Finding No.	IBT Finding	Requirements Not Met
1.	The Redundancy Report is incomplete, as acknowledged in the Redundancy Report itself. Only a limited number of locations have been investigated. The necessary checks have not been comprehensive, and results confirming adequacy have not been sufficiently presented. The required FDF time history analyses have not been included. It fails to include an evaluation of redundancy considering the loss of internal box girder struts, tower table struts, or tie-down stays. Without validation, it states that dynamic effects were not considered for the tower anchor boxes.	TP Section 13.2.1.3 PTI DC45.1-12 § 5.5
2.	Multiple drilled shafts fail to meet the geotechnical axial load resistance and structural flexural resistance. At foundation 1NT, 16 of the 20 drilled shafts are deficient under the required service *and* strength loads. At foundation 1ST, 12 of the 20 drilled shafts are deficient under the required service loads and 10 of the 20 drilled shafts are deficient under the required service loads.	TP Section 13.2.1.14 TxDOT *Geotechnical Manual* Chapter 5 Sections 2 and 3[3] TxDOT *Bridge Design Manual – LRFD* Chapter 2 Section 1[4] AASHTO *LRFD Bridge Design Specifications* Section 10.5.5.2.4[5] AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance.
3.	The use of a 1.33 "overstress" factor is not supported by any current codes and is therefore not allowed.	CDA Section 3.2.1 The use of a 1.33 overstress factor is not permitted by AASHTO *LRFD Bridge Design Specifications* Seventh Edition, as revised through 2015.

[2] Reference is made to the full April 23, 2022 IBT Report, as updated on August 12, 2022, for a complete identification of all deficiencies.
[3] Incorporated by Section 13.1 of the Technical Provisions ("TP").
[4] Incorporated by Section 13.1 of the TPs.
[5] Incorporated by Section 13.1 of the TPs.

4.	Supplemental piling system at foundation 1NT is not considered in the design.	April 3rd Agreement Section 5 Paragraph 1 (Replacement New Harbor Bridge Engineer to review and sign/seal all engineering and Design Work performed by Figg.)
5.	Foundations 1NT *and* 1ST each contain one drilled shaft that is deficient in regard to the required load resistance due to uplift.	AASHTO *LRFD Bridge Design Specifications* Section 5.7.4.5 AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance.
6.	Reinforcement details for the drilled shafts in foundations 1NT *and* 1ST reveal insufficient minimum longitudinal reinforcing.	AASHTO *LRFD Bridge Design Specifications* Sections 5.7.3.4, 5.7.4.2, 5.8.2.4, 5.13.4.5.2 TxDOT *Bridge Detailing Guide* Chapter 7 Section 2[6] TxDOT Standard Drawing FD Common Foundation Details
7.	The foundation caps for foundations 1NT and 1ST do not have sufficient capacity for the required shear load resistance at the location of the tower legs.	AASHTO *LRFD Bridge Design Specifications* Sections 5.13.3.6, 5.8.1.2, 5.8.1.4, 5.8.3.2, 5.8.3.5, 5.7.3.2, and 5.11.2 AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance.
8.	Compressive stress limits are exceeded in the top slab of the superstructure girders adjacent to the pylon	AASHTO *LRFD Bridge Design Specifications* Sections 5.5.1, 5.9.4.2.1 and Table 5.9.4.2.1-1
9.	Demand/Capacity ratios are exceeded under the strength loading case *and* the construction loading case for the superstructure girders.	AASHTO *LRFD Bridge Design Specifications* Sections 5.7.3 and 5.7.4.7 AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance.

[6] Incorporated by Section 13.1 of the TPs.

10.	Insufficient connection at the delta frame and girder bottom cast-in-place joint.	AASHTO *LRFD Bridge Design Specifications* Sections 5.5.1, 5.9.4.2.2
11.	The anchor head at the connection of the delta frame and girder joint is not designed to resist against shear friction force present at this location.	AASHTO *LRFD Bridge Design Specifications* Section 5.10.9.2.3 Unable to validate that this work meets AASHTO specs. CDA Section 3.2.1 (all Design Work and Construction Work shall be in accordance with Good Industry Practice)
12.	The maximum allowable stress within the delta frame diagonal struts is exceeded for the required service loads.	AASHTO *LRFD Bridge Design Specifications* Sections 5.5.1, 5.9.4.2.2
13.	The vertical bursting reinforcement within the delta frame anchors of tendons TD2 and TD3 is insufficient.	AASHTO *LRFD Bridge Design Specifications* Section 5.10.9.3.2 (resistance to vertical bursting forces is not sufficient)
14.	The Type 1 delta frame's bottom strut fails under Extreme III load combination.	AASHTO *LRFD Bridge Design Specifications* Section 5.7.4 AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance. CDA § 3.2.1 (all Design Work and Construction Work shall be in accordance with Good Industry Practice) PTI DC45.112 §§ 5.3, 5.5.
15.	The bearings for towers 1N, 1S, 2N, *and* 2S are insufficient for the uplift load condition.	AASHTO *LRFD Bridge Design Specifications* Section 14.6.1
16.	The vertical stays for towers 1N and 1S are insufficient for the uplift load condition and do not provide redundancy in the loss-of-stay condition.	PTI DC45.1-12: Recommendations for Stay-Cable Design, Testing, and Installation, May 2012, Section 5.3.3 AASHTO *LRFD Bridge Design Specifications* Section 1.3.2.1 requires that demand be less than or equal to resistance.

17.	The torsion resulting from the unbalanced construction loads exceed the torsional cracking moment (T_{cr}) of the girders during erection of the superstructure.	*AASHTO LRFD Bridge Design Specifications* Section 5.14.2.3.3
18.	The principal stresses in the webs of the girders are exceeding during erections of the superstructure.	*AASHTO LRFD Bridge Design Specifications* Section 5.14.2.3.3
19.	There are at least four significant findings related to the Wind Report.	TP Section 13.2.117
20.	The General Notes of the Erection Manual omit critical design information related to the casting and erection of the superstructure.	TP Sections 2.2.7.5.1 and 2.2.7.7 CDA Section 3.2.1
21.	Superstructure cambers are not included in the Erection Manual.	TP Sections 2.2.7.5.1 and 2.2.7.7 Comprehensive Development Agreement Section 3.2.1
22.	The longitudinal and cross-tie steel reinforcement for towers 1N, 1S, 2N, *and* 2S do not meet the detailing requirements for hollow rectangular compression members.	*AASHTO LRFD Bridge Design Specifications* Section 5.10.12

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

TECHNICAL MEMORANDUM

TOWER DRILLED SHAFTS

DOCUMENT NUMBER: TM1001

08/12/2022

Revision 0

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

12 August 2022

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

TECHNICAL MEMORANDUM

TOWER DRILLED SHAFTS

DOCUMENT NUMBER: TM1001

ORIGINATORS: Christopher Hall, PE (CA) and Michael W. Smart, PE

Revision History

Revision	Date	Description
0	08/12/2022	Original Issue

Table of Contents

1. Introduction

This technical memorandum discusses previously reported findings of the Independent Structural Analysis (ISA) group concerning the drilled shafts of the main tower foundations of the Corpus Christi New Harbor Bridge, cable-stay main bridge. The information presented herein demonstrates that the current design of the tower drilled shafts do not meet the project requirements for resisting axial forces subject to Strength limit state demands. These findings show an exceptional amount of concurrent overloading at multiple drilled shafts within the same foundation cap.

The Developer's Lead Engineer (DLE) has presented an alternate approach to evaluate the main tower drilled shafts, which assumes a rigid foundation cap and plastic deformation of the drilled shafts. The ISA has determined these assumptions are not appropriate for a foundation of this arrangement, size, loading, and complexity, which consequently leads to a significant underestimation of the actual loads on the drilled shafts.

This finding has been documented in previous reports (see References 6. and 7. below) and discussed in meetings (see References 8., 9., and 10. below).

The main body of this memorandum provides context, presents a summary of results, and discusses these findings. Relevant supporting calculations are included in the Appendices, along with more in-depth discussion about the DLE's alternate approach.

2. References

The following documents are referenced in this memorandum.

1. Texas Department of Transportation (TxDOT), "Technical Provisions for US 181 Harbor Bridge Project: Comprehensive Development Agreement." ["TP"]
2. American Association of State Highway and Transportation Officials (AASHTO), "LRFD Bridge Design Specifications," 7th Edition, 2014 with 2015 Interim Revisions. ["AASHTO LRFD"]
3. "277609-NHB-PLN-M02-02" ["Design Drawings" or "Current Design"]
4. "277609-NHB-REP-MWER-02: US181 Harbor Bridge Replacement Project: Wind Engineering Report," Revision 2, May 4, 2021. ["Wind Report"]
5. "277609-NHB-REP-New Harbor Bridge Geotechnical Engineering Report, Rev 03" stamped June 28, 2021. ["Geotechnical Report"]
6. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 1010 dated January 8, 2021 ["ISA Phase 1 Report"]
7. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 2010 dated April 23, 2022 ["ISA Phase 2 Part 1 Report"]
8. Meeting Notes of 26 May 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["May 2022 Meeting"]
9. Meeting Notes and Presentations of 10 June 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["June 2022 Meeting"]

10. Meeting Notes and Presentations of 29 July 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT [“July 2022 Meeting”]
11. Meeting Notes of 5 August 2022 virtual meeting between TxDOT, FDLLC, HNTB, ARUP-CFC, BLWTL, and IBT [“August 2022 Meeting”]
12. “Corpus Christi US-181 Harbor Bridge Replacement Project Footing Stiffness in Foundation Group Analysis” prepared by Joseph Juzwin, Ted Zoli, and Matthew Riegal of HNTB dated June 24, 2021 [“HNTB Report”]
13. January 12, 2021 FDLLC Presentation to TxDOT [“January 2021 Presentation”]
14. Foundation 1ST Load Case Provided by the Developer’s Lead Engineer received August 5, 2022 [“DLE Foundation Loads”]
15. TxDOT/HNTB Review Comments spreadsheet file: Master_Sub-4403_CRF(2021-06-09)Rev05.xlsx dated January 12, 2021 [“TxDOT/HNTB Review Comments”]

3. Background

The foundation elements that support the main towers are identified as 1NT and 1ST. Foundation 1NT includes a total of 19, 10’ϕ drilled shafts and 1, 8’ϕ drilled shafts (middle test shaft is not part of the functional foundation). Foundation 1ST includes a total of 20, 10’ϕ drilled shafts. The tower is skewed to the foundation by a ~37.4° angle, which leads to a non-orthogonal positioning between the tower and foundation shaft group. The foundation cap is 132’-0” wide, 72’-0” long, and 18’-0” deep.

See Figures 1 and 2 below for the specific layout of the drilled shafts.

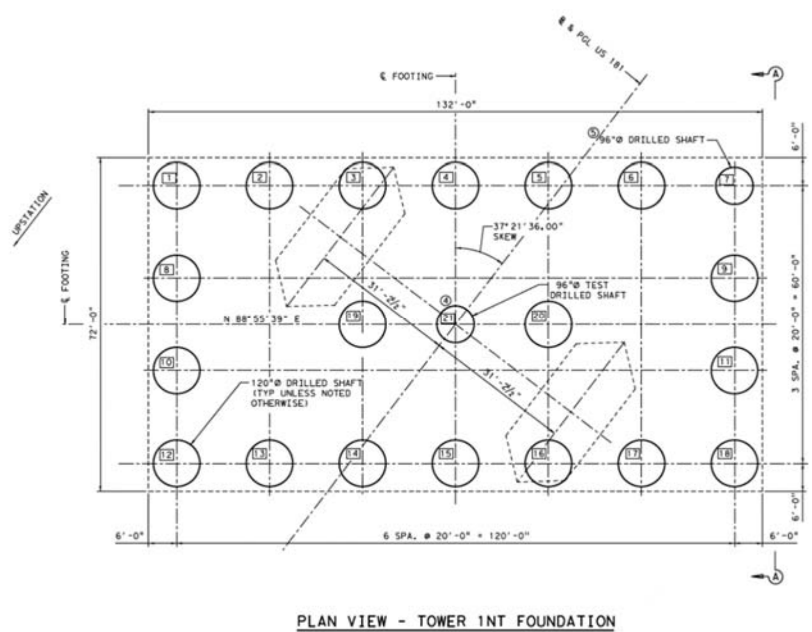


Figure 1: Foundation Cap 1NT, Foundation Cap and Shaft Layout, Drawing NHB-30A (by Others)

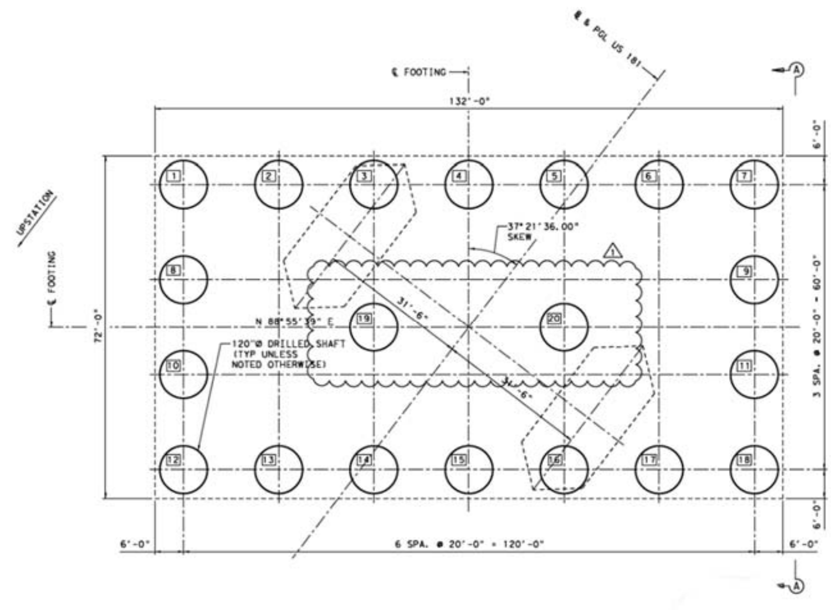


Figure 2: Foundation Cap 1ST, Foundation Cap and Shaft Layout, Drawing NHB-30B (by Others)

For all the analyses 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 6. above).

AASHTO LRFD §10.5.1, §10.5.5.2.4, and §10.8.3.5 specify the requirements for computing the geotechnical axial capacity of drilled shafts to resist the calculated demands. This technical memorandum focuses on the geotechnical axial capacity under compression Strength loading. Using the code provisions listed above, the ISA identified multiple drilled shafts that are under capacity, indicating a serious deficiency in the foundation design.

The geotechnical capacities of the drilled shafts were calculated by the ISA’s geotechnical consultant and are included in the ISA Phase 1 Report. The capacities calculated by the ISA’s geotechnical consultant agree reasonably with capacities presented in the Developer’s Geotechnical Report (see Reference 5 above).

The drilled shaft loads are calculated by applying external tower forces to soil-structure interaction analysis models that includes the foundation cap, drilled shafts, and soil properties, using software that is specially designed for this task.

The appendices of this design memorandum show applicable and summarized calculations. The calculations include the updated (Rev. 2) Wind Report loadings. Appendix A presents the drilled shaft loading summary for two of the critical load cases on the foundations as calculated by the ISA. Additionally, the drilled shaft demands using loads recently provided by the DLE are presented (see

Reference 14). Appendix B includes a technical review of the DLE's approach to the design of the foundations.

As required by the Technical Provisions, TP §13.2.1.4, "The New Harbor Bridge shall be designed with an operational importance factor of 1.05. The operational importance factor shall be applied to the superstructure, including stay cables, and the towers." The tower foundations are intended to have the 1.05 factor applied, as it is a critical part of the towers' structural system. However, in the July 2022 Meeting, the 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, this technical memorandum presents calculations both with and without the 1.05 importance factor. Although the Technical Provisions intended the tower foundations to be considered important, neglecting the 1.05 importance factor does not change the conclusions discussed herein.

4. Summary of Findings

The calculations in the appendices demonstrate that the tower foundation drilled shafts at 1NT and 1ST, as currently designed, do not meet the requirements of AASHTO LRFD §1.3.2.1, which specify that demand be less than or equal to capacity under axial loading. The following results are found:

- Axial capacity of the drilled shafts per AASHTO LRFD §10.5.5.2.4 and §10.8.3.5:
  - Foundation 1NT Factored ISA Geotechnical Capacity, 10' ϕ Shaft = 13,200 kips
  - Foundation 1NT Factored DLE Geotechnical Capacity, 10' ϕ Shaft = 13,300 kips
  - Foundation 1ST Factored ISA Geotechnical Capacity, 10' ϕ Shaft = 15,100 kips
  - Foundation 1ST Factored DLE Geotechnical Capacity, 10' ϕ Shaft = 15,400 kips

- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.05 and 90-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 17,159 kips, **D/C = 1.30**
  - Foundation 1ST, ISA Maximum Shaft Load = 18,431 kips, **D/C = 1.22**
  - Foundation 1ST, DLE Maximum Shaft Load = 17,857 kips, **D/C = 1.18**

- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.00 and 90-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 16,667kips, **D/C = 1.26**
  - Foundation 1ST, ISA Maximum Shaft Load = 17,926 kips, **D/C = 1.19**
  - Foundation 1ST, DLE Maximum Shaft Load = 17,418 kips, **D/C = 1.15**

- Axial Strength load demands for drilled shaft pile group with 1.05 Importance Factor – Individual Load Case and 90-deg wind:
  - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **8**
  - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **7**
  - Foundation 1ST, DLE Loads, # of Drilled Shafts with D/C > 1.00 = **5**

- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.05 and 270-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 17,484 kips, **D/C = 1.32**
  - Foundation 1ST, ISA Maximum Shaft Load = 17,951 kips, **D/C = 1.19**
  - DLE loads not provided for 270-deg wind

- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.00 and 270-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 16,960 kips, **D/C = 1.28**
  - Foundation 1ST, ISA Maximum Shaft Load = 17,502 kips, **D/C = 1.16**
  - DLE loads not provided for 270-deg wind

- Axial strength load demands for drilled shaft pile group with 1.05 Importance Factor – Individual Load Case and 270-deg wind:
  - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **10**
  - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **6**

- Axial strength load demands for drilled shaft pile group with 1.05 Importance Factor – Enveloped Load Cases (include wind loads at 90-deg and 270-deg):
  - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **17**
  - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **13**

Note: The D/C ratio above designates a Demand-to-Capacity ratio, where values above 1.00 do not meet the AASHTO LRFD §1.3.2.1 requirement.

See Appendix A for a summary of the design approach and summary of axial loads. It is noted that the DLE has provided load combinations only for foundation 1ST and with wind loading in the 90-deg direction, and therefore 1NT and 1ST with 270-deg wind loads were not available for comparison in this technical memorandum.

The ISA team has reviewed the DLE's approach to the design of the tower foundations at 1NT and 1ST, which included the assumption of a rigid cap and plastic behavior of the drilled shafts. The ISA team concludes that such assumptions are not appropriate for these foundations. The ISA team agrees with the HNTB Report dated June 24, 2021 (see Reference 12 above) that refutes the rigid cap assumption and cautions that such a simplification is inaccurate, unconservative, and potentially dangerous for the design of the Harbor Bridge towers. The technical evaluation described herein support this position. See Appendix B for more discussion about the ISA team's review of the DLE's approach to the design of the tower foundations.

5. Conclusion

DRILLED SHAFT CAPACITY

The axial capacity of the drilled shafts is exceeded at the tower foundations caps under Strength limit state loadings. The applicable requirements include AASHTO LRFD §1.3.2.1, §10.5.1, §10.5.5.2.4, and

§10.8.3.5. The worst-case loading of a single drilled shaft indicates the demand exceeding its capacity by 32%.

It is also important to emphasize the breadth of the overloading condition. Under a single load case, 10 of the 20 drilled shafts simultaneously exceed their load capacity at 1NT, and 7 of the 20 drilled shafts simultaneously exceed their load capacity at 1ST. When an envelope of the applied loads that includes both 90-deg and 270-deg is considered (e.g. wind changing direction, which occurs during hurricanes), the total quantity of overloaded drilled shafts total 17 at 1NT (85% of shafts fail) and 13 at 1ST (65% of shafts fail). This is indicative of a foundation drilled shaft group that is exceedingly deficient to resist AASHTO LRFD design loadings.

Lastly, the drilled shaft loadings were investigated considering a range of assumptions to evaluate sensitivity, including consideration of a load case provided by the DLE for the Tower at 1ST. For every case considered over this range of possibilities, the maximum drilled shaft demand significantly exceeded capacity.

RIGID CAP/PLASTIC SHAFT ANALYTICAL APPROACH

The procedures to rationalize the drilled shaft foundation design were described by the DLE by referencing a presentation provided to TxDOT (January 2021 Presentation), which was subsequently presented to the ISA at the June 2022 meeting. This presentation was also cited in response to TxDOT/HNTB's review of the foundation submittal (TxDOT/HNTB Review Comments – Item ID No. 15 – see Reference 15. above), which remains an unresolved item in this comment log.

In Appendix B, the ISA team has evaluated the rigid cap simplification and plastic shaft alternate analytical approach and has reached the same conclusion as TxDOT/HNTB: that these assumptions are not appropriate for the 1NT and 1ST foundations.

SUMMARY OF CONCLUSIONS

The ISA demonstrated the following findings concerning the current design of tower foundation drilled shafts:

- The drilled shafts were found to be loaded significantly above their design capacity.
- Multiple drilled shafts were found to be simultaneously overloaded under a single load case.
- An envelope of load cases revealed that most of the drilled shafts in the 1NT and 1ST tower foundations would not have adequate capacity, considering only 2 of the 24 wind angles (90-deg and 270-deg wind load cases.)
- The drilled shaft overloads were identified considering ISA demands, as well as demands for 1ST recently provided by the DLE.
- The above findings were demonstrated whether the 1.05 importance factor was considered or neglected.
- The DLE's approach to designing the drilled shafts is not consistent with the actual behavior of the foundations, which have been load tested. The application of two assumptions in this approach – "rigid cap" and "plastic shaft behavior" – significantly underestimates the actual

loading in the drilled shafts. The assumption of a rigid foundation is not consistent with AASHTO Chapter 4. The ISA Team's evaluation of such assumptions is consistent with TxDOT/HNTB Review Comments (See Reference 15. above) in responses to earlier submittals and the HNTB Report (see Reference 12).

- The current design of the tower foundation drilled shafts does not meet the project requirements. The following are applicable requirements:
  - TP §13.2.1.4
  - AASHTO LRFD §1.3.2.1
  - AASHTO LRFD §4
  - AASHTO LRFD §10.5.1

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 drilled shafts. 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:

1. Plastic Deformation of Drilled Shafts, Pgs 3 to 15: We have reviewed the commentary code reference identified in the DLE presentation, AASHTO LRFD §C10.8.3.5, and the deflection limit state identified is not applicable to a plastic load design.

   As clearly stated in the clause, "…it is customary to establish the failure criterion at the strength limit state at a gross deflection equal to five percent of the base diameter for drilled shafts…" and it goes on to say, "…For consistency in the interpretation of both static load test (Article 10.8.3.5.6) and the normalized curves of Article 10.8.2.2.2."

   Noting that this load test failure criterion is in the commentary and is specifically used for setting the gross deflection for load tests, it is not designated for use as a design parameter. In fact, the requirements of AASHTO LRFD §10.8.3.5 "Nominal Axial Compression Resistance of Single Drilled Shafts" specifically identifies the strength criteria for an individual drilled shaft to be solely force based.

Appendix A. Summary of Drilled Shaft Loads at 1NT and 1ST

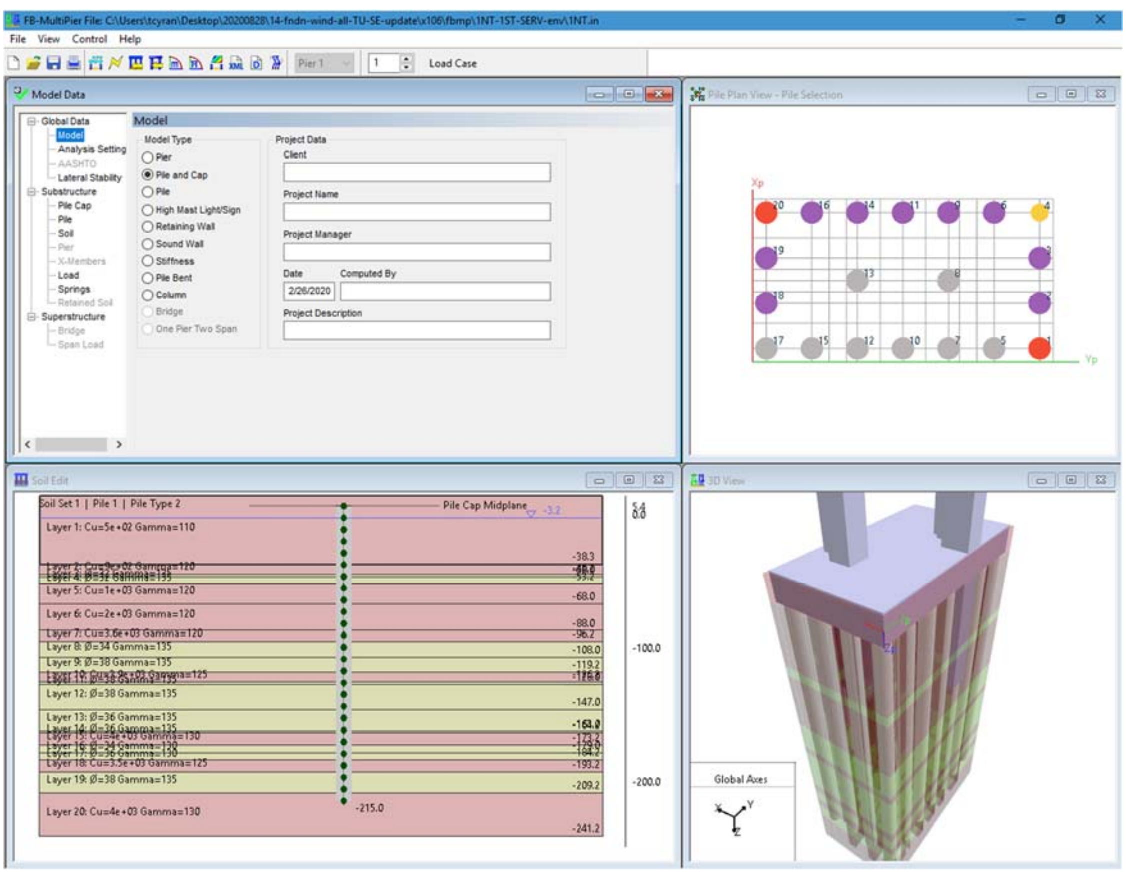
A.1 General Analysis Model Overview

The ISA modeling approach for the main tower foundations is documented in the Phase 1 reports. For reference, a general review is presented below.

The local analysis of the drilled shafts at foundations 1NT and 1ST is conducted using FB-MultiPier, a 3-D computer soil-structure interaction program that utilizes:

- Non-linear structural finite elements
- Non-linear static soil models
- Axial side friction resistance
- Axial tip resistance
- Lateral resistance
- Torsional resistance

A screenshot from FB-MultiPier showing an isometric view of the footing, drilled shafts, and soil layers at foundation 1NT is provided below:



The loads generated by the tower legs are applied per their concomitant pairs.

A.2 Input Loads

The ISA design investigation of the foundations includes a thorough application of the AASHTO defined load combinations, considering the updated (Rev 2) Wind Report with various wind attack angles (at 15° increments) and various dynamic effect combination possibilities (24 possibilities per wind angle increment).

Based on recent technical discussions with the DLE, their primary wind load direction is generally from the 90-deg angle, so for comparison purposes and for brevity, only this wind direction and its reverse direction (270-deg) are presented herein.

The ISA load combinations for these two directions are as follows:

90-Deg Wind Load Combinations

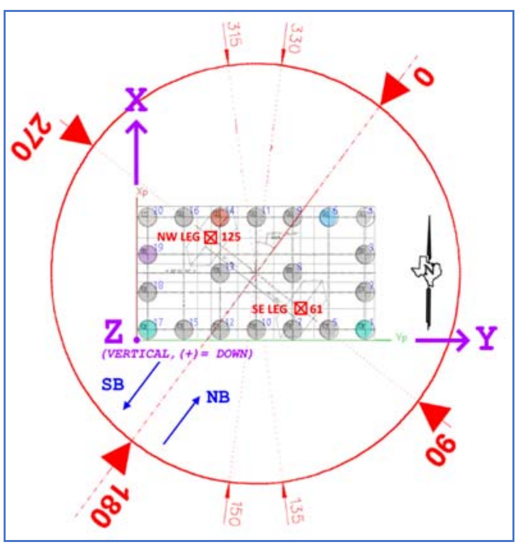
STRENGTH 3 LOAD COMBINATIONS -- IN DIRECTION OF FOUNDATION (SEE FIGURE BELOW)

		IMPORTANCE	NORTH-WEST (NW) LEG, NODE 125						SOUTH-EAST (SE) LEG, NODE 61 (CONCOMITANT)					
ANALYSIS	TOWER	FACTOR	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
ISA	1NT	1.05	10,570	-10,777	112,164	-217,740	-492,147	-59,962	-318	4,831	59,469	-150,597	-609,022	-15,268
ISA	1NT	1	10,067	-10,264	106,823	-207,371	-468,711	-57,107	-303	4,601	56,637	-143,426	-580,021	-14,541
ISA	1ST	1.05	11,666	-10,143	112,714	-54,166	-726,753	12,308	538	5,852	60,869	4,862	-801,565	103,721
ISA	1ST	1	11,111	-9,660	107,347	-51,587	-692,146	11,722	512	5,573	57,971	4,630	-763,395	98,782

270-Deg Wind Load Combinations

STRENGTH 3 LOAD COMBINATIONS -- IN DIRECTION OF FOUNDATION (SEE FIGURE BELOW)

		IMPORTANCE	NORTH-WEST (NW) LEG, NODE 125 (CONCOMITANT)						SOUTH-EAST (SE) LEG, NODE 61					
ANALYSIS	TOWER	FACTOR	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft
ISA	1NT	1.05	-673	-5,676	57,095	-590,597	781,256	6,555	-11,615	10,138	113,920	49,071	703,456	-76,738
ISA	1NT	1	-641	-5,406	54,376	-562,473	744,053	6,243	-11,062	9,655	108,495	46,734	669,958	-73,084
ISA	1ST	1.05	339	-4,783	57,667	145,890	605,521	75,608	-10,509	10,751	113,693	205,071	483,210	37,131
ISA	1ST	1	323	-4,555	54,921	138,943	576,687	72,008	-10,009	10,239	108,279	195,306	460,200	35,363



For comparison, the DLE provided the following load combination as one of their critical cases from a 90-deg direction at the 1ST foundation.

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STRENGTH 3 LOAD COMBINATION

LOAD COMBINATION	NORTH-WEST (NW) LEG, NODE 125						SOUTH-EAST (SE) LEG, NODE 61 (CONCOMITANT)						IMPORTANCE
	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	FX kip	FY kip	FZ kip	MX kip-ft	MY kip-ft	MZ kip-ft	FACTOR
STR 3 COMBINATION	11,678	-9,801	110,073	-37,898	-749,025	-83,668	578	6,154	63,083	614,999	-824,469	6,208	1.05
ARUP STR3 COMBINATION	11,170	-9,342	111,900	248,900	-404,600	-78,890	-858	6,920	62,970	191,000	-486,400	29,140	1.0

The ISA load combination above was for a 120-deg wind angle, which maximizes a foundation cap loading. As noted earlier, a 90-deg angle combination and a 270-deg angle combination are presented herein, which better compares with the load case provided by the DLE.

As evident from the load combinations, there is an appreciable difference between the longitudinal flexural moment results for the individual tower leg loads. However, as presented in the August 2022 Meeting, this moment does not have a significant influence on the drilled shaft maximum loads. Both the ISA loads and DLE's loads were applied to the same model, as presented in the following section.

A.3 Drilled Shaft Loads

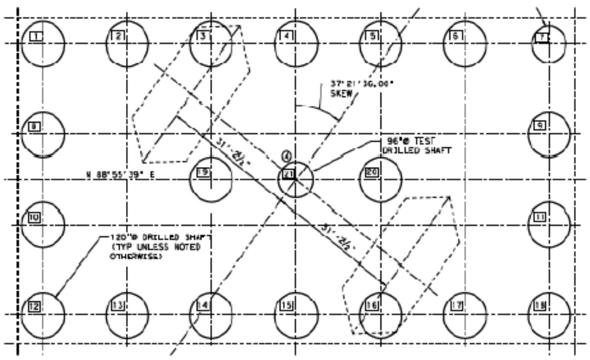
As noted in the body of this technical memorandum, the DLE has argued that the 1.05 importance factor should not apply to the tower foundations. The foundations are a critical part of the tower structural system, and so the 1.05 importance factor should be applied. However, neglecting the importance factor does not impact the calculations enough to overcome the deficiencies associated with this finding, so the following combinations, that both consider the importance factor (Factor = 1.05) and neglect the importance factor (Factor = 1.00), are presented:

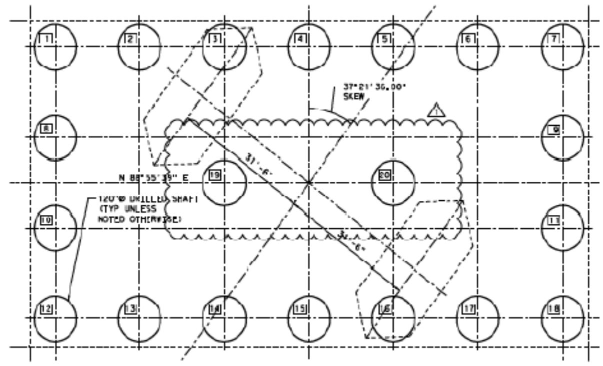
- ISA STR 3 Load Combination, 90-Deg Wind, Foundation 1NT/1ST, Factor 1.05
- ISA STR 3 Load Combination, 90-Deg Wind, Foundation 1NT/1ST, Factor 1.00
- DLE STR 3 Load Combination, 90-Deg Wind, Foundation 1ST, Factor 1.05
- DLE STR 3 Load Combination, 90-Deg Wind, Foundation 1ST, Factor 1.00
- ISA STR 3 Load Combination, 270-Deg Wind, Foundation 1NT/1ST, Factor 1.05
- ISA STR 3 Load Combination, 270-Deg Wind, Foundation 1NT/1ST, Factor 1.00

Plots showing the drilled shaft axial loads along with demand-to-capacity ratios (D/C) for each of the cases above are shown on the following pages.

The factored Strength capacities for the drilled shafts are 13,200 kips for foundation 1NT and 15,100 kips for 1ST. The geotechnical axial capacities were calculated independently by the ISA's Geotechnical Engineer and were provided in the Phase 1 report. The D/C ratios in the Figures below are calculated with these values.

Figure A1: ISA 90-deg Wind STR 3 Load Combination – 1.05 Factor





1NT 90-degree, wind mode 24, IF = 1.05

1ST 90-degree, wind mode 23, IF = 1.05

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

-16,794	-17,088	-17,159	-16,745	-15,742	-14,335	-9,173
-15,207						-10,063
		-14,446		-13,177		
-12,113						-6,069
-4,043	-6,198	-7,742	-8,433	-6,741	-2,697	-123

1NT

-17,704	-18,241	-18,431	-17,987	-16,816	-14,993	-11,698
-15,762						-9,640
		-15,194		-13,959		
-11,029						-6,030
-1,337	-4,045	-6,317	-7,569	-6,201	-2,705	-101

1ST

CASE 0: D/C RATIO

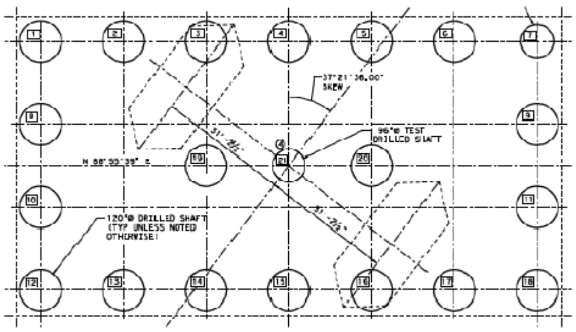
1.27	1.29	1.30	1.27	1.19	1.09	0.91
1.15						0.76
		1.09		1.00		
0.92						0.46
0.31	0.47	0.59	0.64	0.51	0.20	0.01

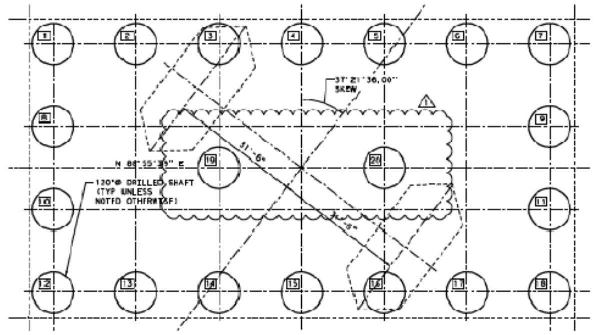
1NT

1.17	1.21	1.22	1.19	1.11	0.99	0.77
1.04						0.64
		1.01		0.92		
0.73						0.40
0.09	0.27	0.42	0.50	0.41	0.18	0.01

1ST

Figure A2: ISA 90-deg Wind STR 3 Load Combination – 1.00 Factor





1NT

90-degree, wind mode 24, IF = 1

1ST

90-degree, wind mode 23, IF = 1

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

-16,187	-16,563	-16,667	-16,214	-15,162	-13,612	-8,542
-14,585						-9,426
		-13,856		-12,648		
-11,439						-5,779
-3,652	-5,845	-7,441	-8,169	-6,676	-3,039	-126

1NT

-17,078	-17,709	-17,926	-17,502	-16,254	-14,208	-10,832
-15,018						-8,996
		-14,574		-13,314		
-10,205						-5,786
-1,192	-3,871	-6,089	-7,341	-6,134	-2,957	-115

1ST

CASE 0: D/C RATIO

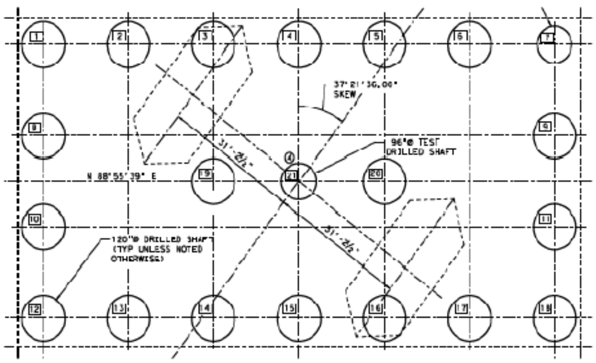
1.23	1.25	1.26	1.23	1.15	1.03	0.85
1.10						0.71
		1.05		0.96		
0.87						0.44
0.28	0.44	0.56	0.62	0.51	0.23	0.01

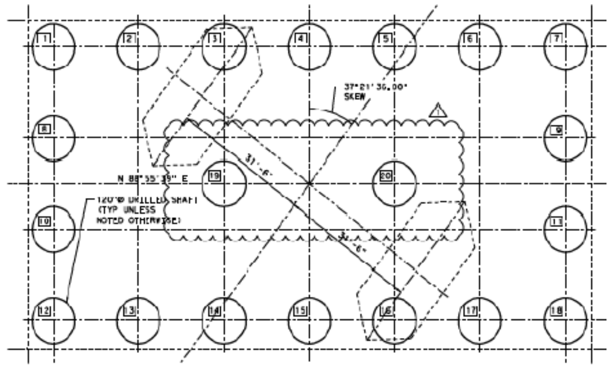
1NT

1.13	1.17	1.19	1.16	1.08	0.94	0.72
0.99						0.60
		0.97		0.88		
0.68						0.38
0.08	0.26	0.40	0.49	0.41	0.20	0.01

1ST

Figure A3: DLE 90-deg Wind STR 3 Load Combination – 1.05 Factor





1NT

90-degree, IF = 1.05

1ST

90-degree, IF = 1.05

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

1NT

-16,567	-17,478	-17,857	-17,564	-16,367	-14,380	-10,964
-14,574						-9,694
		-14,996		-14,006		
-10,053						-7,470
-1,589	-5,453	-8,099	-9,582	-8,933	-6,269	-2,216

1ST

CASE 0: D/C RATIO

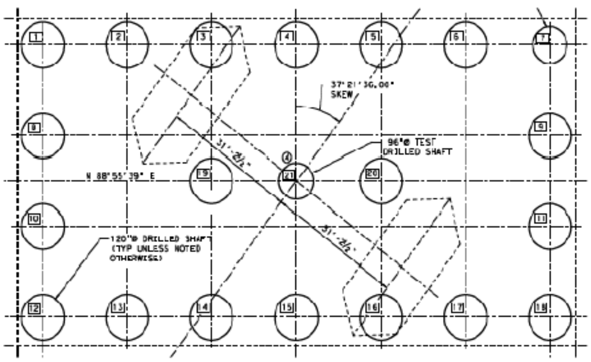
N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

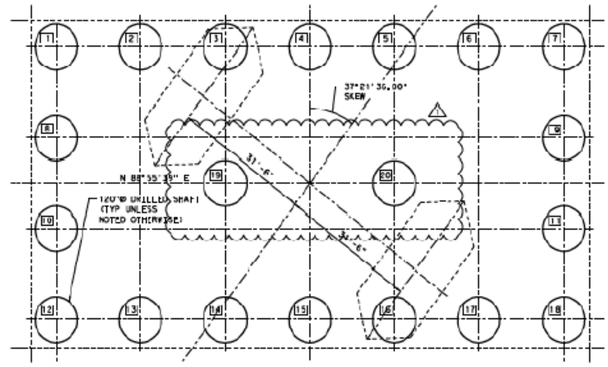
1NT

1.10	1.16	1.18	1.16	1.08	0.95	0.73
0.97						0.64
		0.99		0.93		
0.67						0.49
0.11	0.36	0.54	0.63	0.59	0.42	0.15

1ST

Figure A4: DLE 90-deg Wind STR 3 Load Combination – 1.00 Factor





1NT

90-degree, IF = 1

1ST

90-degree, IF = 1

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

1NT

-15,968	-16,918	-17,418	-17,065	-15,832	-13,605	-10,191
-13,749						-9,118
		-14,418		-13,419		
-9,358						-7,151
-1,524	-5,233	-7,792	-9,264	-8,696	-6,199	-2,438

1ST

CASE 0: D/C RATIO

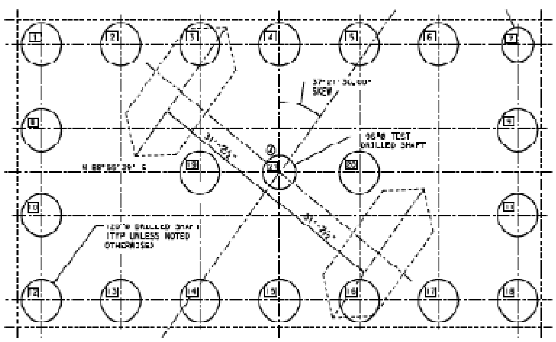
N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

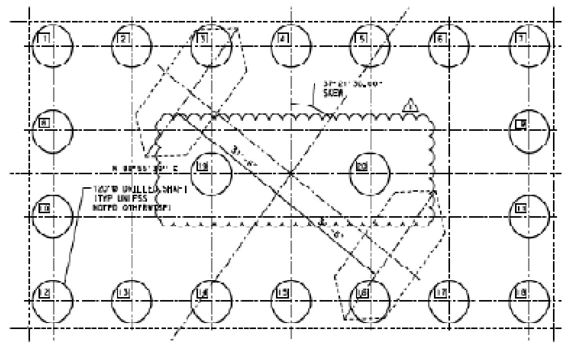
1NT

1.06	1.12	1.15	1.13	1.05	0.90	0.67
0.91						0.60
		0.95		0.89		
0.62						0.47
0.10	0.35	0.52	0.61	0.58	0.41	0.16

1ST

Figure A5: ISA 270-deg Wind STR 3 Load Combination– 1.05 Factor





1NT 270-degree, wind mode 2, IF = 1.05

1ST 270-degree, wind mode 9, IF = 1.05

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

1NT

-133	-2,659	-5,125	-6,088	-5,251	-3,688	-913
-8,179						-11,767
		-13,440		-14,427		
-11,625						-15,307
-13,388	-15,045	-16,238	-17,106	-17,484	-17,386	-17,073

1ST

-108	-3,443	-7,065	-8,492	-7,477	-5,587	-3,116
-4,967						-11,386
		-13,236		-14,927		
-7,977						-15,535
-9,713	-13,647	-16,060	-17,435	-17,951	-17,803	-17,322

CASE 0: D/C RATIO

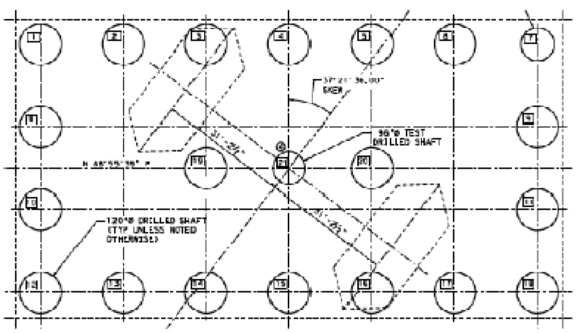
1NT

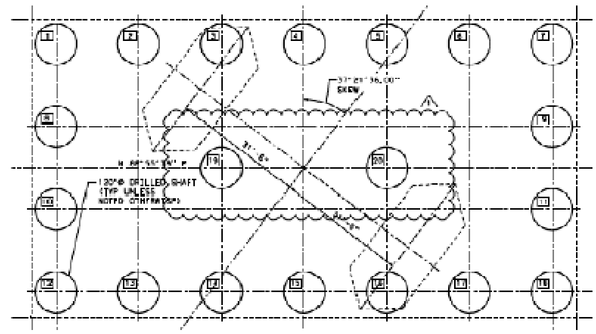
0.01	0.20	0.39	0.46	0.40	0.28	0.09
0.62						0.89
		1.02		1.09		
0.88						1.16
1.01	1.14	1.23	1.30	1.32	1.32	1.29

1ST

0.01	0.23	0.47	0.56	0.50	0.37	0.21
0.33						0.75
		0.88		0.99		
0.53						1.03
0.64	0.90	1.06	1.15	1.19	1.18	1.15

Figure A6: ISA 270-deg Wind STR 3 Load Combination– 1.00 Factor





1NT — 270-degree, wind mode 2, IF = 1

1ST — 270-degree, wind mode 9, IF = 1

CASE 0: MAX COMPRESSION IN SINGLE DRILLED SHAFT (CONCURRENT RESULTS), KIPS

1NT

-136	-2,949	-5,136	-5,925	-5,063	-3,498	-739
-7,698						-11,020
		-12,869		-13,797		
-10,883						-14,671
-12,568	-14,381	-15,641	-16,591	-16,960	-16,836	-16,483

1ST

-125	-3,574	-6,917	-8,209	-7,187	-5,323	-2,861
-4,798						-10,634
		-12,652		-14,315		
-7,465						-14,850
-8,986	-12,908	-15,444	-16,899	-17,502	-17,306	-16,679

CASE 0: D/C RATIO

1NT

0.01	0.22	0.39	0.45	0.38	0.27	0.07
0.58						0.83
		0.97		1.05		
0.82						1.11
0.95	1.09	1.18	1.26	1.28	1.28	1.25

1ST

0.01	0.24	0.46	0.54	0.48	0.35	0.19
0.32						0.70
		0.84		0.95		
0.49						0.98
0.60	0.85	1.02	1.12	1.16	1.15	1.10

Appendix B. ISA Review of Developer's Lead Engineer Design Approach for Foundations

B.1 General Overview

The ISA and Developer's Lead Engineer (DLE) participated in technical meetings to discuss the ISA's findings showing design deficiencies, which included overloading of the drilled shafts at the Tower Foundations 1NT and 1ST (see References 8., 9., 10., and 11).

The DLE indicated the same issue was raised by TxDOT and their Construction Engineering and Inspection firm (HNTB), and discussions were held where the design approach for the foundations was discussed. The TxDOT and HNTB comments were tracked and after multiple iterations of comments, the issue was considered unresolved (See Reference 15, Comment #15).

At the June 2022 Meeting, the DLE provided a presentation from January 12, 2021 (see Reference 13 - January 2021 Presentation) that explained the DLE's approach to design of the drilled shafts, which assumes a rigid foundation cap and drilled shafts with axial capacities that are capped at their allowable geotechnical capacity.

The information on this topic also includes a technical letter provided by TxDOT and HNTB, which documents their opposition to this approach, with supporting technical references (See Reference 12).

B.2 ISA Technical Assessment

The ISA Team's role is to assess the design using independent analyses and calculations of the structure. Its role is not to review the DLE's calculations. However, for this tower foundation drilled shafts finding, a review of the DLE's approach was performed by the ISA in an attempt to reconcile differences between the ISA and DLE. It is noted that the ISA's calculations considered the structural stiffness of the cap in their foundation analyses, and the ISA used a consistent modeling approach to calculate both the drilled shaft geotechnical loads and their structural capacity. The ISA's findings have identified multiple drilled shafts with loads that significantly exceed the geotechnical axial capacity.

The analytical approach adopted by the DLE is unusual and inappropriate for a foundation of the size, orientation, loading, and complexity of the New Harbor Bridge tower foundations, noting the following observations:

- The assumption of a rigid cap is contrary to the actual behavior of the cap. The analysis models clearly show that the cap has significant flexibility resulting in a load distribution to the drilled shafts that is not accurately captured by rigid cap modeling.
- The use of multiple analysis modeling assumptions decouples the analytical accuracy. In order to use detailed stand-alone models for specific structural elements (e.g. foundations), it is important that there is consistency with the assumed stiffnesses of the global model foundations.
- To choose the most favorable results from multiple models (rigid cap, flexible cap, or non-linear cap) is not accepted practice in engineering. The use of multiple modeling

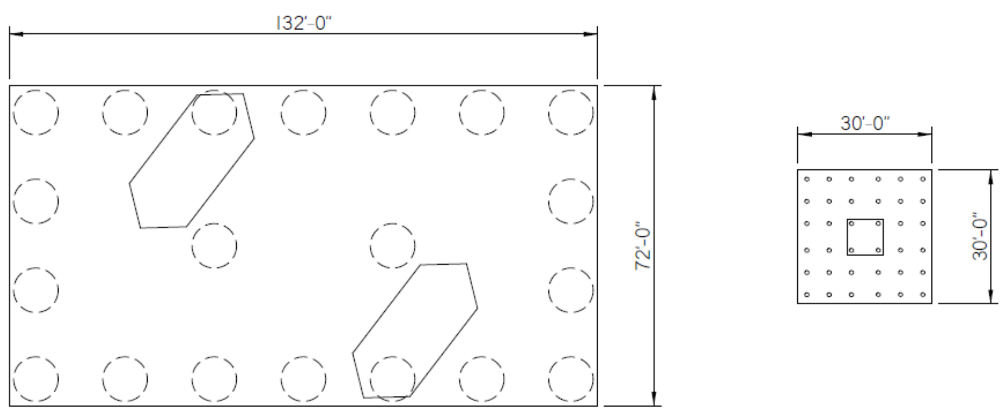
assumptions to envelope potentially uncertain behavior is acceptable; however, every element of the structure must be shown to have adequate capacity for the results of each model.

- The DLE assumes plastic behavior of the drilled shafts using the reduced resistance capacity, which will not match actual behavior.
- The DLE describes the analytical procedure of assuming a rigid cap with plastic drilled shaft behavior as a coupled assumption, but these are stand-alone considerations, neither of which is consistent with the actual behavior of the cap and drilled shafts.
- The DLE rationalized their rigid foundation cap assumption by citing a technical paper from Duan and McBride. This is examined in further detail in the discussion below.

Duan and McBride Analysis

At the June 2022 Meeting, the DLE explained that the technical paper written by Duan and McBride justifies using rigid cap assumptions for the main towers and encouraged the ISA to consider this reference as well.

Upon review of this technical paper, the ISA identified the simplicity of the content of the study. The paper studies a single foundation type, with a single column in the middle of the foundation, with a regular pattern of small diameter piles. It is perfectly symmetric, and it is loaded uniaxially (axial load with bending in only the transverse direction). It is dissimilar from the 1NT and 1ST foundations in almost every applicable variable. A figure with consistent scale between the foundations is provided below showing the differences in plan.



The paper studies three different thicknesses for the 30'x30' plan cap; 1', 3', and 5' deep. The authors of the paper then perform a parametric study using a plate model of the foundation with the 3 different plate thicknesses, and then plot the deformations of the caps and pile loading to visually assess if the results appear linear. The authors did not provide any parameters to quantify the rationale between assessing a cap as rigid or flexible, but rather it appears the approach is qualitative based on a visual assessment of the plot results.

If the determining qualitative observations of this paper were applied to the 1NT and 1ST foundations, the shaft load distribution most closely resembles the non-rigid results presented in the paper. The figures below were taken from Duan and McBride and show plots of the pile loads under the different cap thicknesses. In the non-rigid behavior, a dishing effect in the pile load distribution is clear.

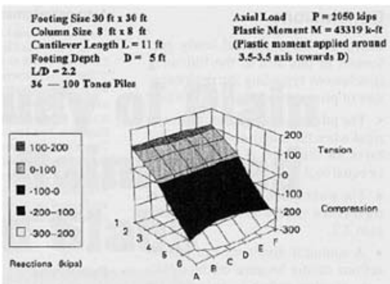


Fig. 6 — Pile reactions of pile cap with depth $D = 5$ ft.

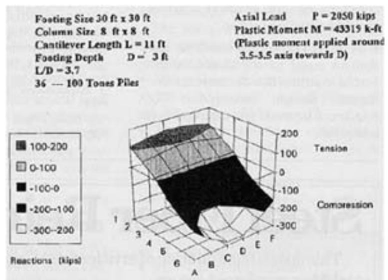


Fig. 7 — Pile reactions of pile cap with depth $D = 3$ ft.

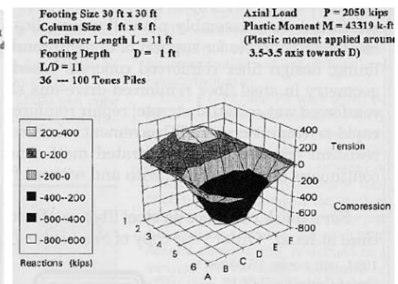
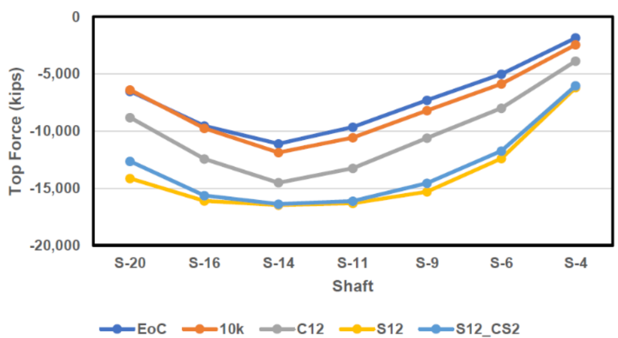
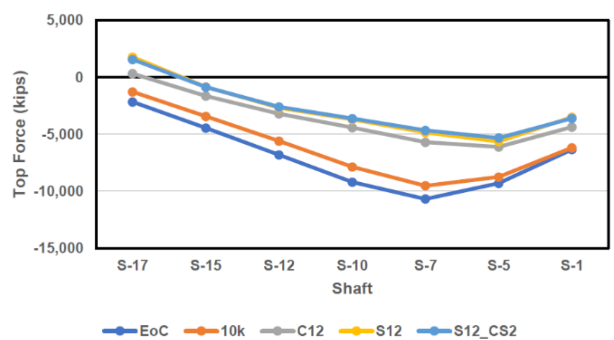


Fig. 8 — Pile reactions of pile cap with depth $D = 1$ ft.

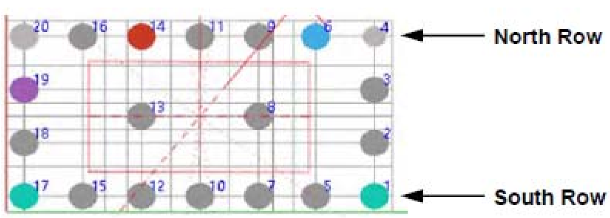
Using the results from the ISA Phase 1 report, similar shaft load plots are generated from a detailed FLAC model of the foundation with soil non-linearity that accurately captures the interaction between soil, shafts, and foundation cap. These plots are shown below for multiple load stages at 1NT.



North Row



South Row



LEGEND:
EoC End of construction
10k 10,000 days
C12 Load combination #12
C12_CS2 Load combination #12, shafts with cracked section to elevation -60 feet

The concentration of the shaft load distribution occurs under the tower legs, and the same dishing behavior documented in the Duan and McBride paper can be visually identified in these plots. In addition, the dishing is asymmetric within the foundation and resembles Figures 7 and 8 of the Duan and McBridge paper. Clearly the criteria to classify a non-rigid foundation in the paper applies to the 1NT and 1ST foundations.

Secondly, the force distribution shown in the plots show the shafts directly under, and adjacent to, the tower legs taking on larger loads and with a similar distribution from initial loading through final strength loading. Assuming these shafts go fully plastic is not accurate, and will appreciably underestimate the actual loading.

B.3 Conclusions

Based on the evaluation above, the ISA does not find the rigid cap and plastic drilled shafts as assumed by the DLE is applicable to the 1NT and 1ST towers, and the ISA agrees with the TxDOT/HNTB technical response provided earlier.

The Duan and McBride study cited by the DLE presents a simplification that may be applicable for conventional design. For the New Harbor Bridge tower foundations, using the same simplifying formula with a more reasonable assumption about the distance between the tower leg to the edge or corner of the cap, this paper actually demonstrates that a rigid cap assumption is not valid for these foundations. The HNTB Report cites both the Duan and McBride study and another study by Ghali, and it demonstrates that the simplifying formulae proposed by both studies would invalidate a rigid cap assumption for the New Harbor bridge tower foundations.

In addition to being an invalid assumption, the rigid cap simplification is unnecessary, given the availability of analytical tools that consider cap stiffness now routinely used for both conventional and unconventional foundation designs. The rigid cap assumption results in non-conservative loading that greatly underestimates actual demands in the tower foundations' drilled shafts.

Concurrently, the DLE's use of a factored reduction of the shaft capacity as the plastic limit is not a valid application of the AASHTO LRFD specifications, and it also results in an underestimation of the shaft demands. It has little correlation with a drilled shaft's actual behavior, which has been load tested and validated.

These two invalid assumptions likely explain why the DLE finds that the current design of the drilled shafts is acceptable and meets the project requirements. However, the ISA calculations, considering appropriate models about foundation cap stiffness and drilled shaft geotechnical behavior, demonstrate that the opposite is true.

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

TECHNICAL MEMORANDUM

TOWER FOUNDATION CAP

DOCUMENT NUMBER: TM1002

08/12/2022

Revision 0

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

12 August 2022

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

TECHNICAL MEMORANDUM

TOWER FOUNDATION CAP

DOCUMENT NUMBER: TM1002

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

Revision History

Revision	Date	Description
0	08/12/2022	Original Issue

Table of Contents

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.

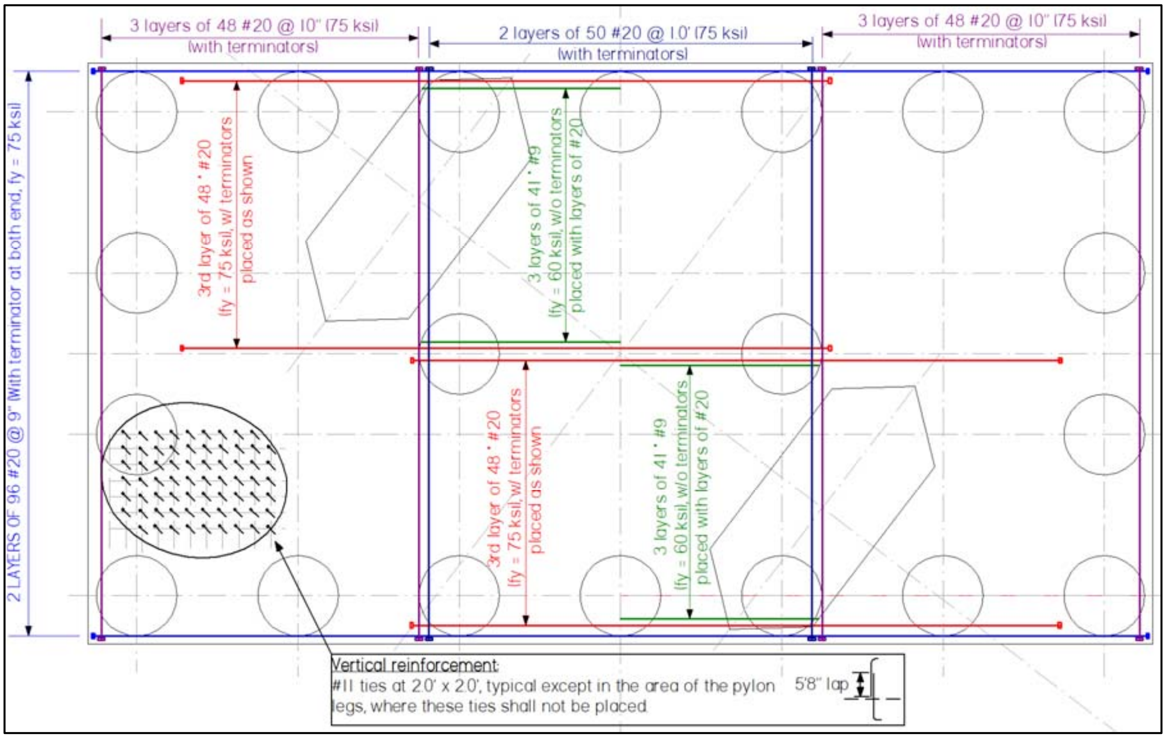


Figure 1: Foundation Cap 1ST, Bottom Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

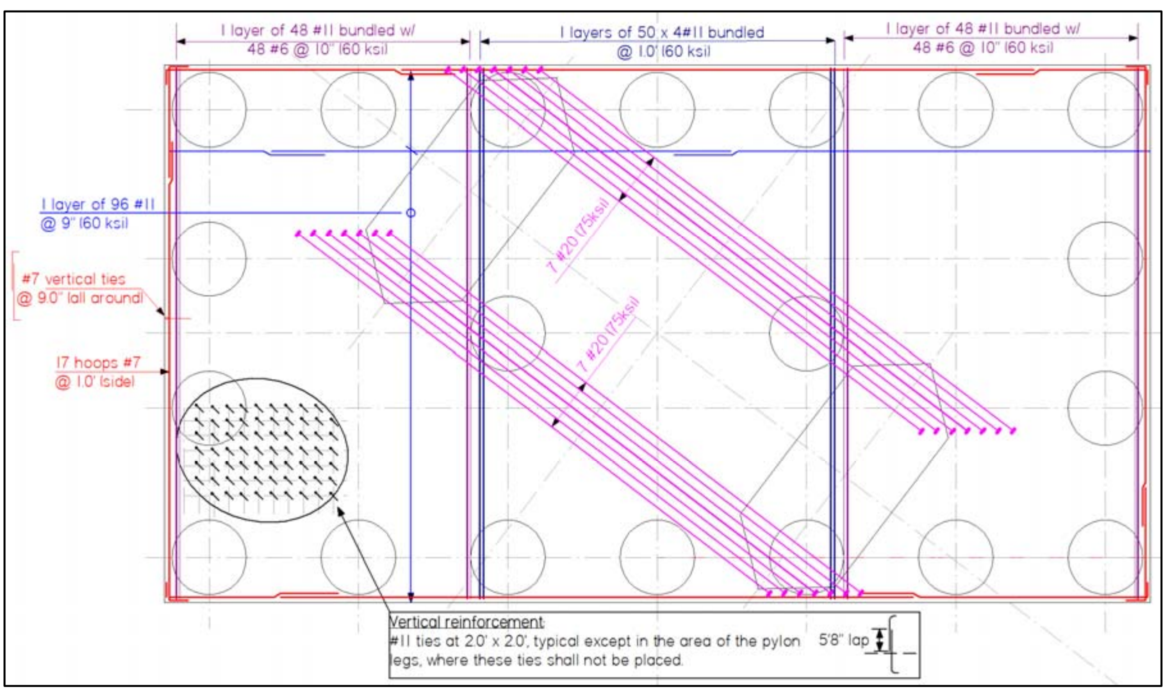


Figure 2: Foundation Cap 1ST, Top Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

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

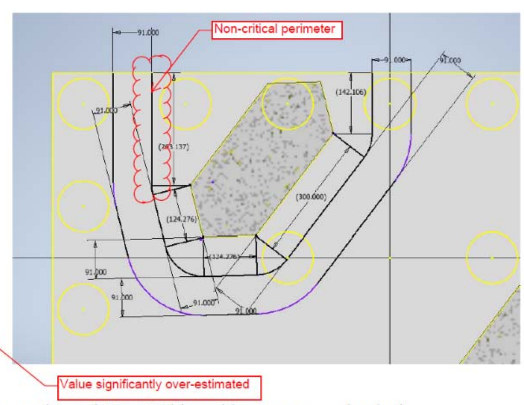
Just prior to issuing this Technical Memorandum, the ISA received design input from the DLE related to the foundation cap. While the content in other sections of this memorandum was completed prior, we provide the following technical comments in response to the DLE's presentation:

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**



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. Appendix A1 – Plate Element Finite Element Model for Evaluation of Bending Demand and One-Way Action

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).

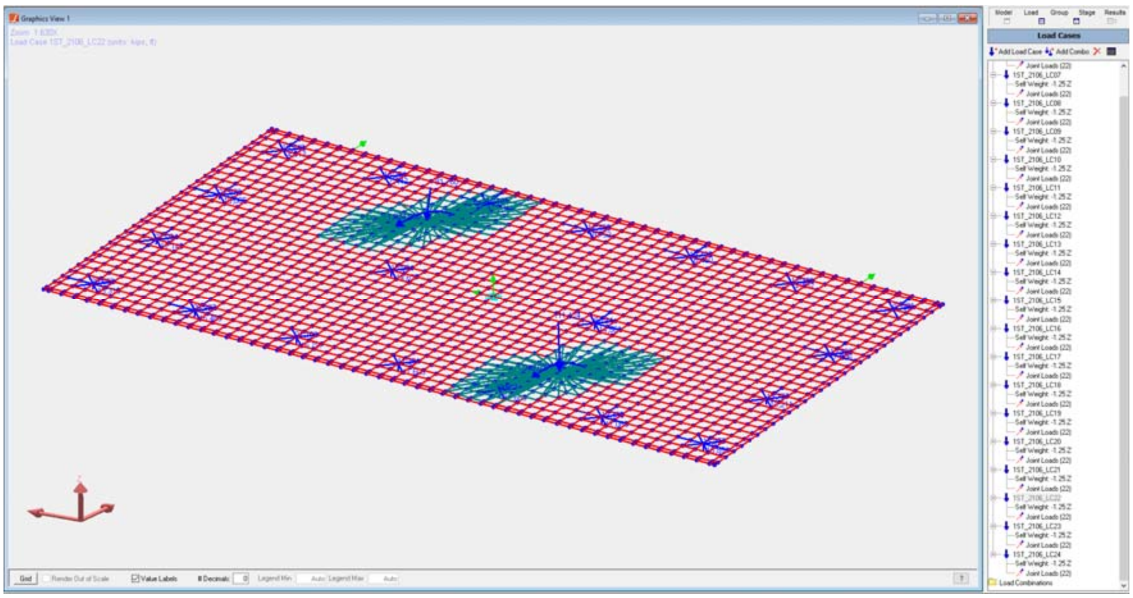


Figure 3: FEM Model of Tower Foundation Cap 1ST (1NT similar)

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.

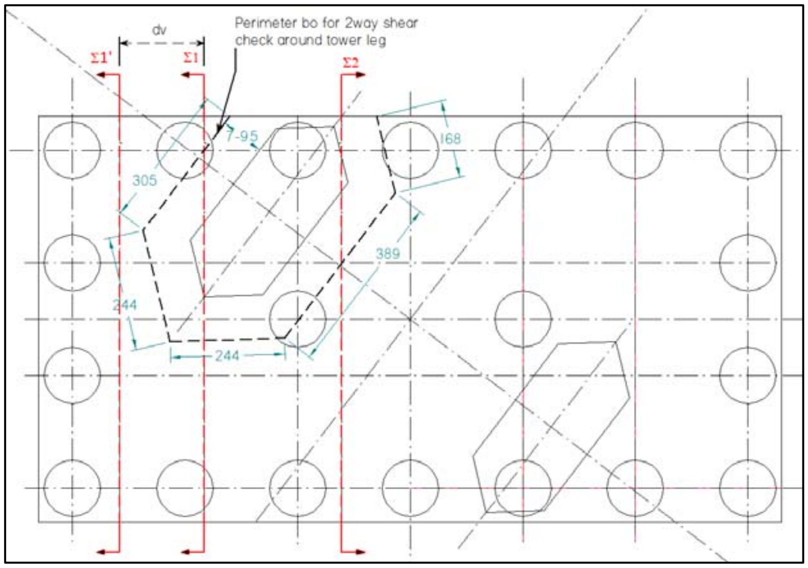


Figure 4: Critical Section Definition for Bending (Σ_2), One-Way Action (Σ_1') and Two-Way Action (Perimeter b_o)

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

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.

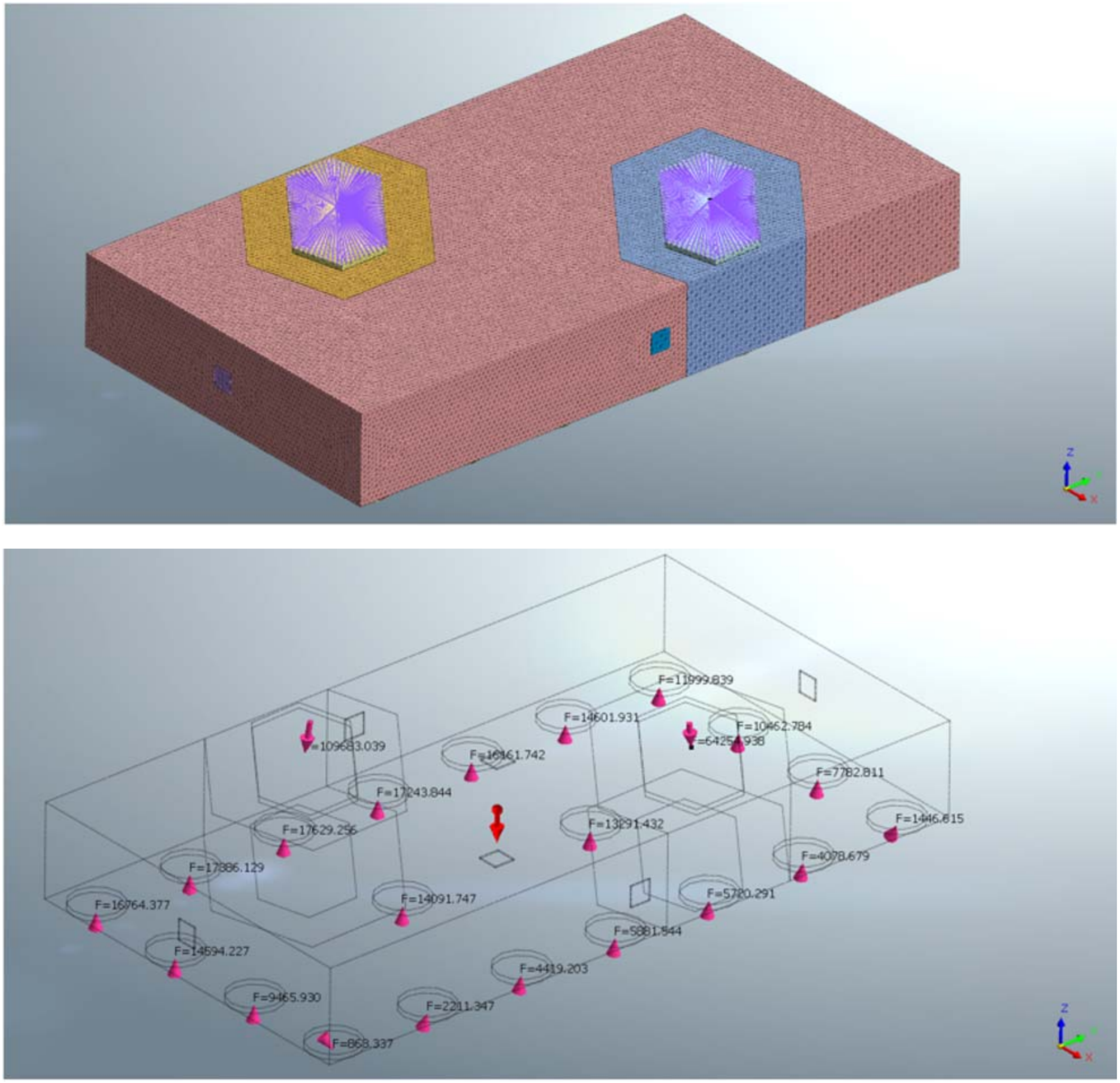


Figure 5: MIDAS FEA NX Brick Element Model - Geometry

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

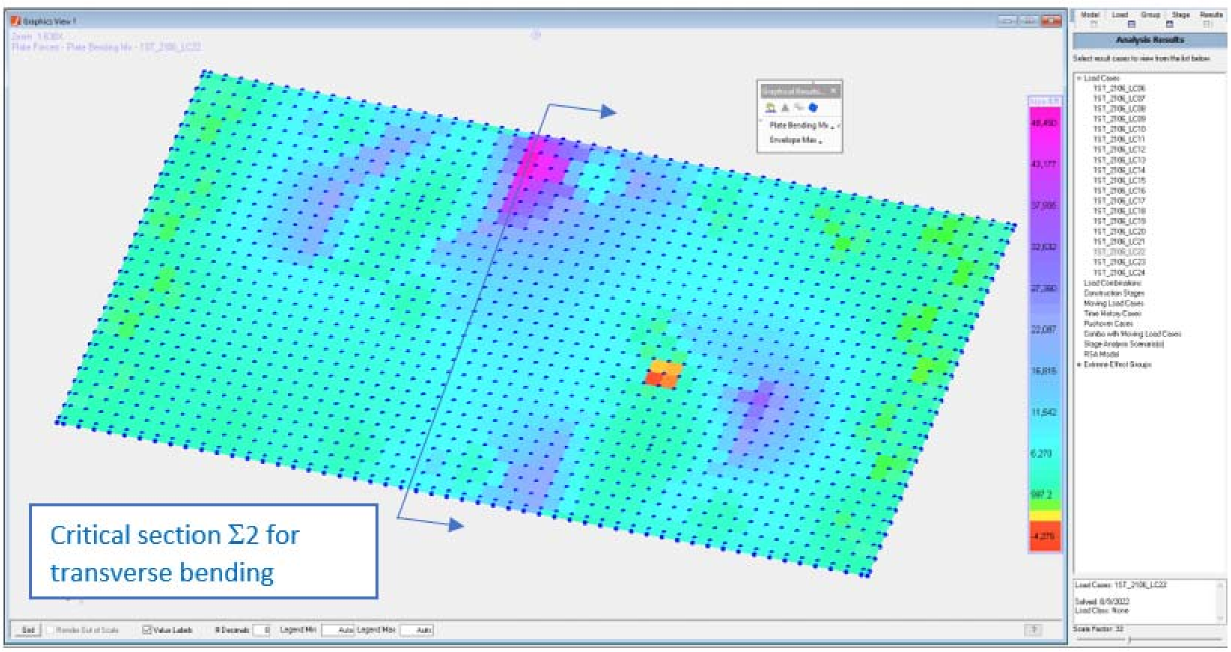
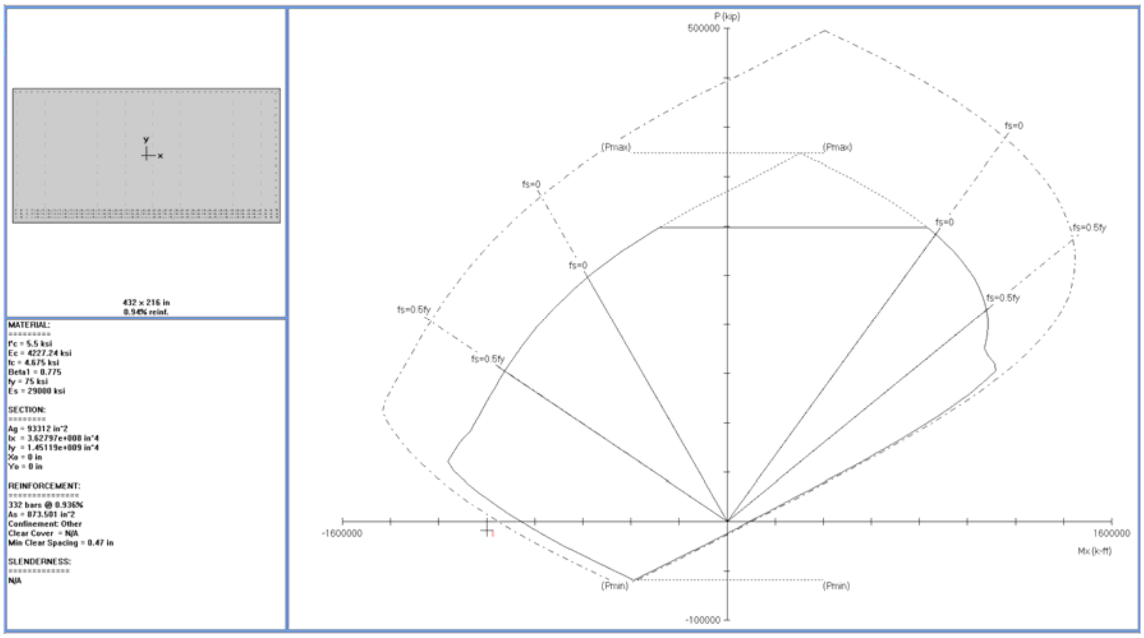


Figure 6: Transverse Bending Demand under Governing Strength III Load Combination (η_I = 1.05)

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

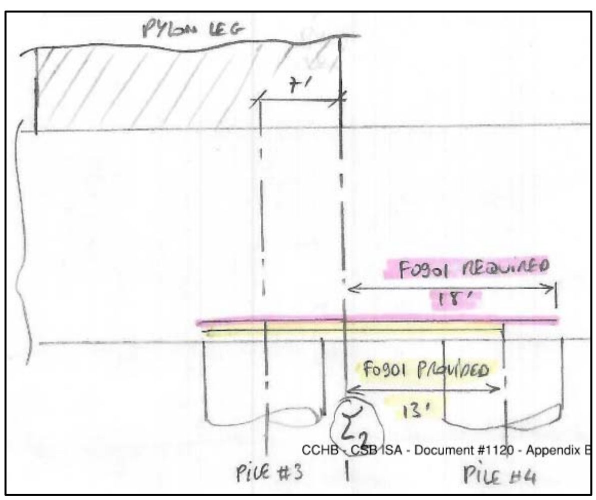


Figure 8: Insufficient Development of Bar F0901 Past Section Σ2, per AASHTO LRFD §5.8.3.5

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

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 ; $V_c = 0.0316\, \beta\, \mathrm{sqrt}(f'c)\, b_v\, d_v$ = 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."*

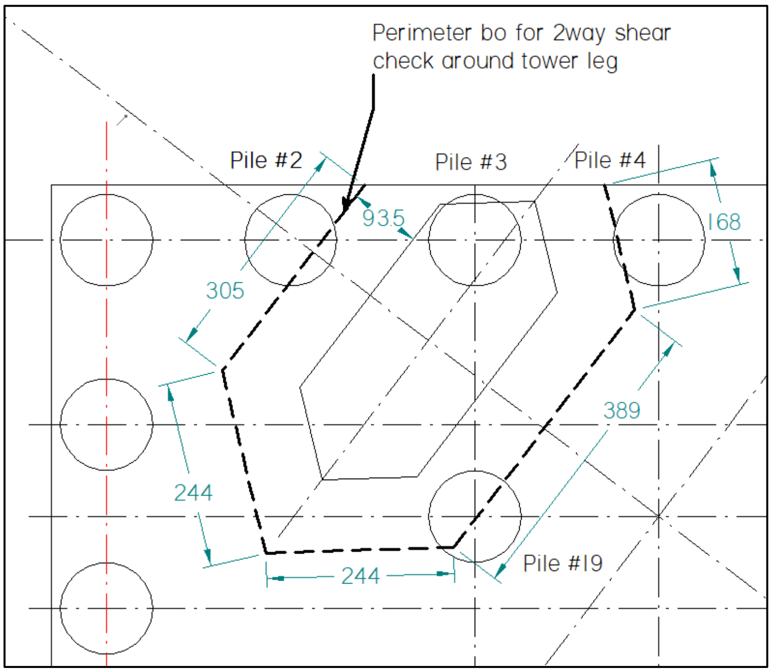


Figure 9: Detail of Critical Perimeter b_o for 2-Way Action: Dimension, Distance to Tower Leg, Intersection with Drilled Shafts.

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

	N-W leg, node 125						S-E leg, node 61 (loads concomitant with loads on 125)					
Load combination	**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.

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

Shear capacity from concrete: V_c = 0.0632 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 =ϕ (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 ϕ 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.

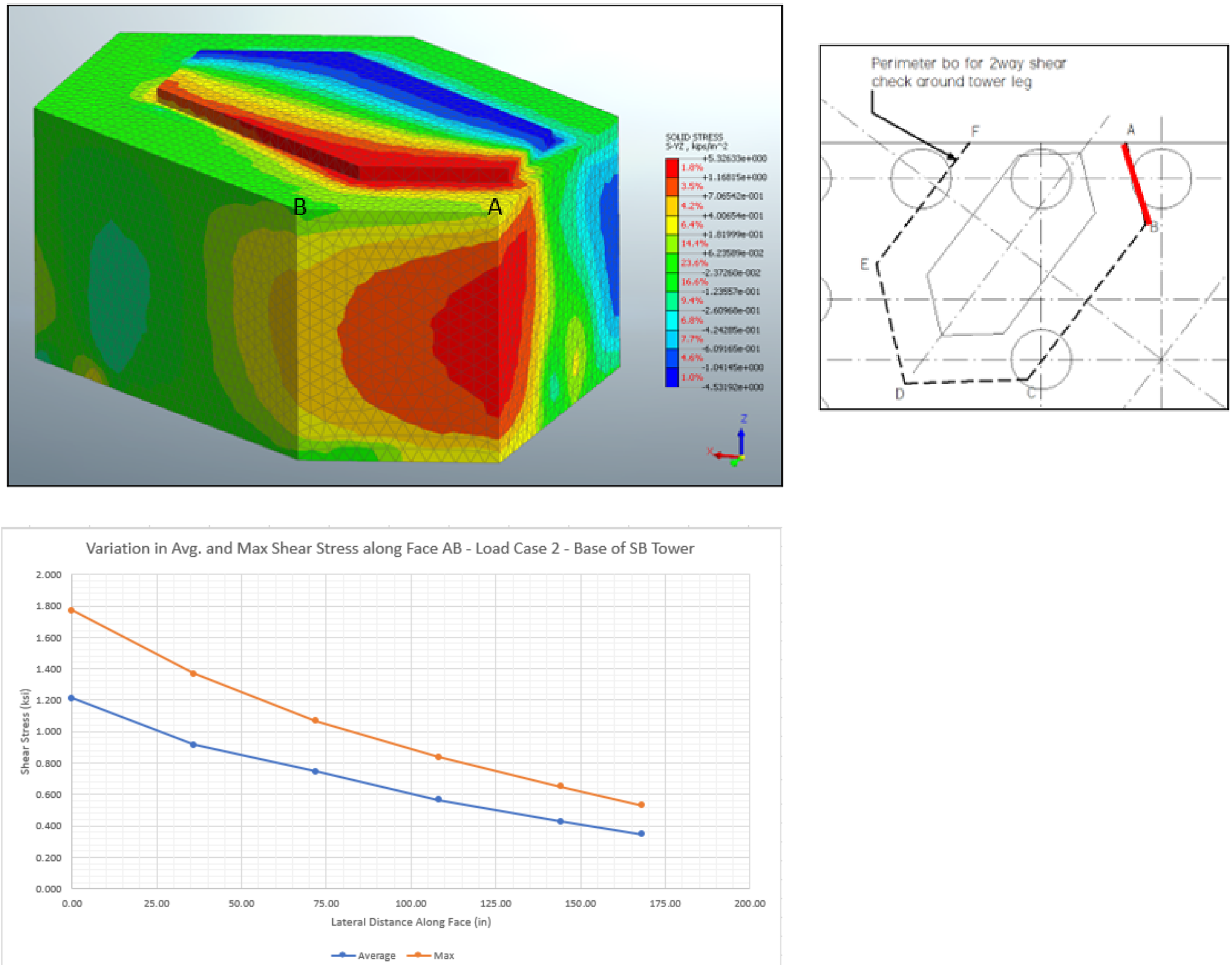
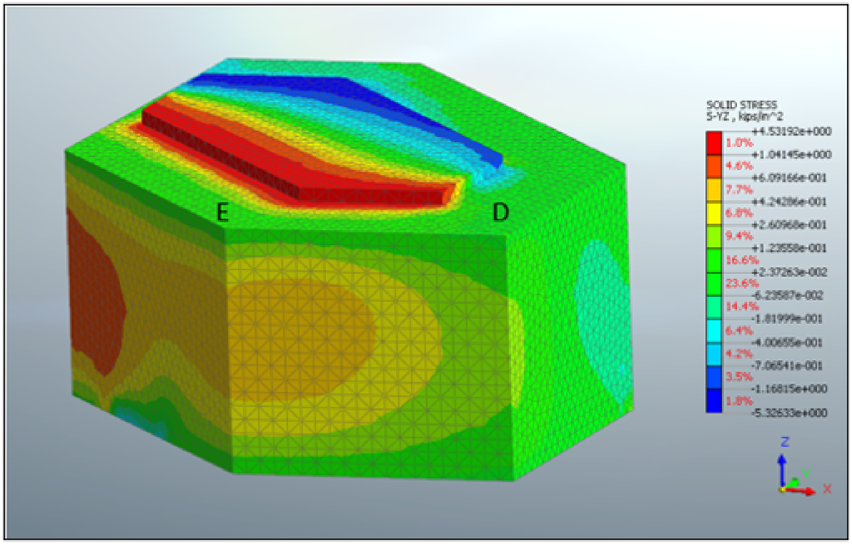
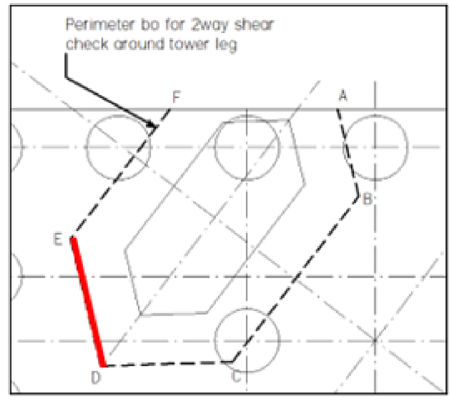


Figure 10: Shear Stress Distribution Along Plane AB for ISA Governing STR III Load Combination





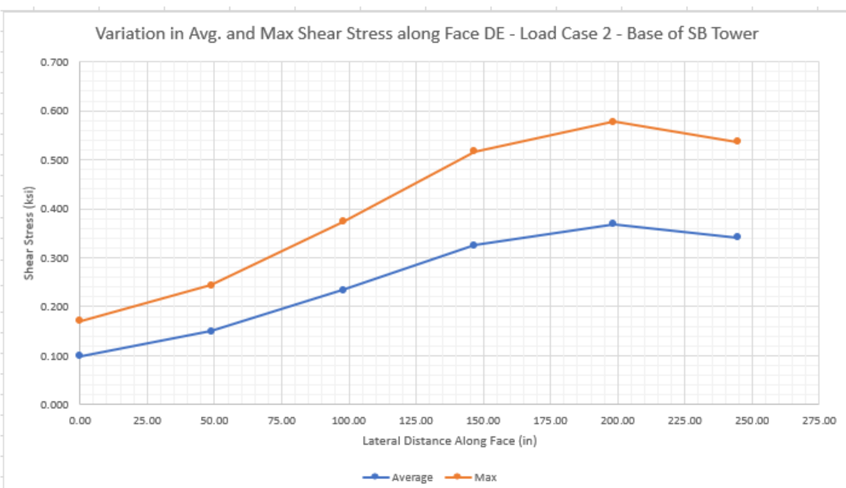
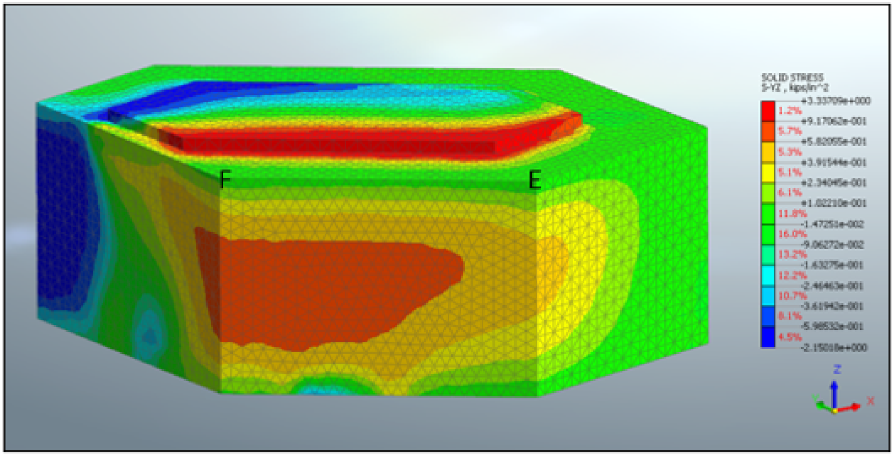
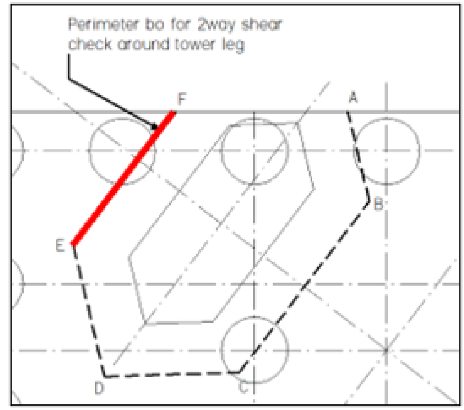


Figure 11: Shear Stress Distribution Along Plane DE for ISA Governing STR III Load Combination





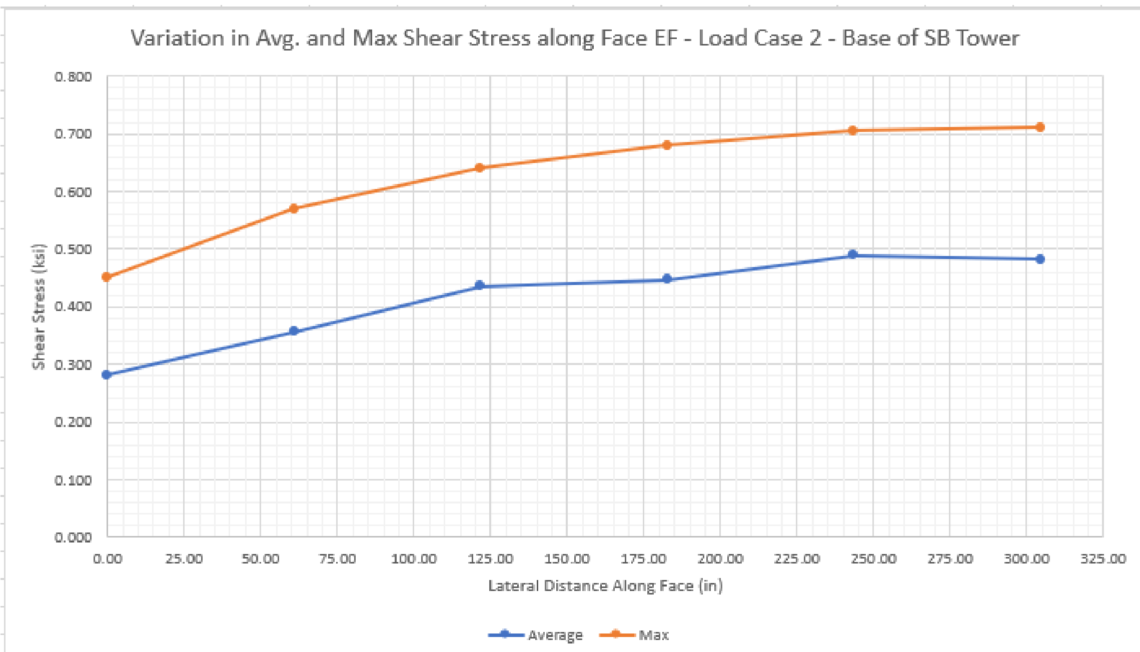


Figure 12: Shear Stress Distribution Along Plane EF of Perimeter bo for ISA Governing STR III Load Combination

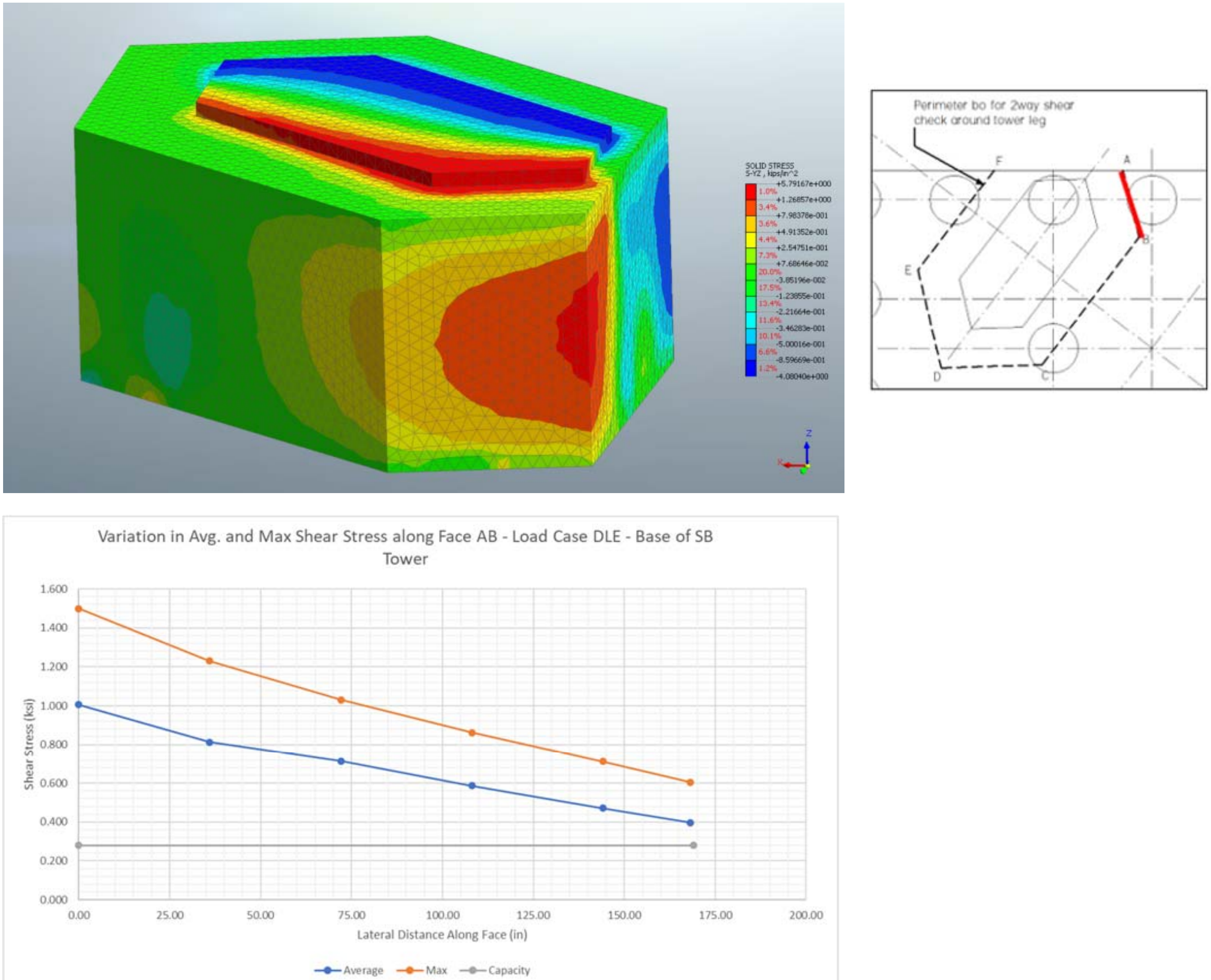


Figure 13: Shear Stress Distribution Along Plane AB of Perimeter b_o Evaluated by the ISA Team Based on DLE Foundation Loads

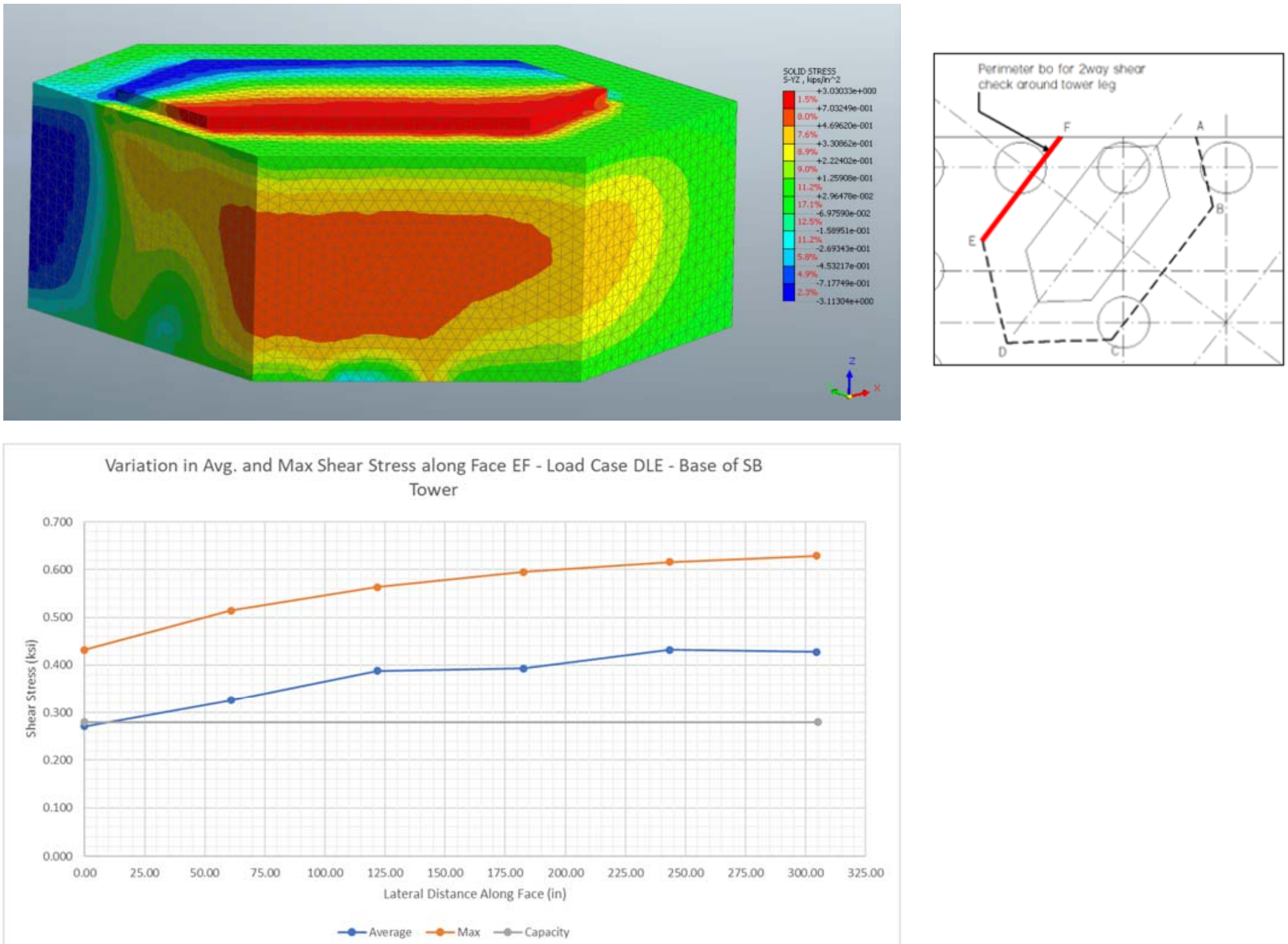


Figure 14: Shear Stress Distribution Along Plane EF of Perimeter b_o Evaluated by the ISA Team Based on DLE Foundation Loads

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

TECHNICAL MEMORANDUM

DELTA FRAME – TO – BOX GIRDER CONNECTION

DOCUMENT NUMBER: TM1003

08/12/2022

Revision 0

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

12 August 2022

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

TECHNICAL MEMORANDUM

DELTA FRAME – TO – BOX GIRDER CONNECTION

DOCUMENT NUMBER: TM1003

ORIGINATOR: Michael W. Smart, PE

Revision History

Revision	Date	Description
0	08/12/2022	Original Issue

Table of Contents

1. Introduction

This technical memorandum discusses previously reported findings of the Independent Structural Analysis (ISA) concerning the delta frame – to – box girder connections of the Corpus Christi New Harbor Bridge, cable-stayed main bridge, where the current design does not meet the project requirements. This and several other findings related to the delta frames have been documented in previous reports (see References 3. and 4. below) and discussed in meetings (see References 6., 7., and 8. below). These findings have yet to be resolved by the Developer, and they have not been addressed by changes in wind input recently observed in the updated (Rev. 2) Wind Report (see Reference 5. below). This technical memorandum focuses solely on the findings related to the delta frame – to – box girder connection design, as shown in Figure 1 below.

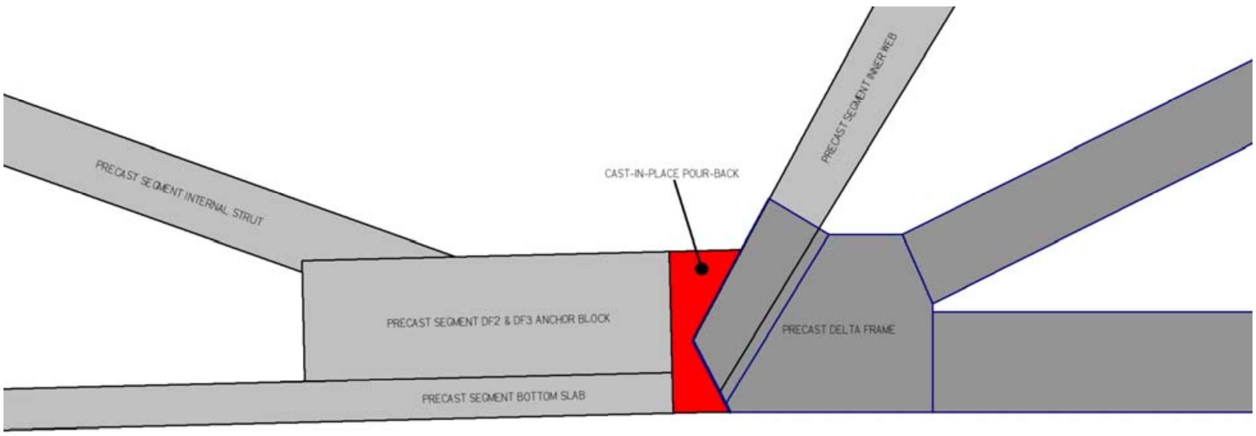


Figure 1: Delta Frame – To- Box Girder Connection (NB Shown, SB Similar)

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-MSUPER_A-00” and “NDC No. 0512” [“Design Drawings” or “Current Design”]
3. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 1010 dated January 8, 2021 [“ISA Phase 1 Report”]
4. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 2010 dated April 23, 2022 [“ISA Phase 2 Part 1 Report”]
5. “277609-NHB-REP-MWER-02: US181 Harbor Bridge Replacement Project: Wind Engineering Report,” Revision 2, May 4, 2021 (First received by ISA Team June 7, 2022) [“Wind 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. "SEL-000772" Letter from FDLLC to TxDOT dated June 24, 2022 ["FDLLC Letter"]

3. Summary of Findings

AASHTO LRFD §5.8.4.1 requires, "The minimum area of interface shear reinforcement specified in Article 5.8.4.4 shall be satisfied." In the current design as shown in Figure 2 below, there has been no interface reinforcement provided across the interfaces between the cast-in-place pour-back and the bottom precast segment DF2 & DF3 anchor block nor between the pour-back and the precast delta frame. This does not meet the requirements of AASHTO LRFD §5.8.4.1 and §5.8.4.4.

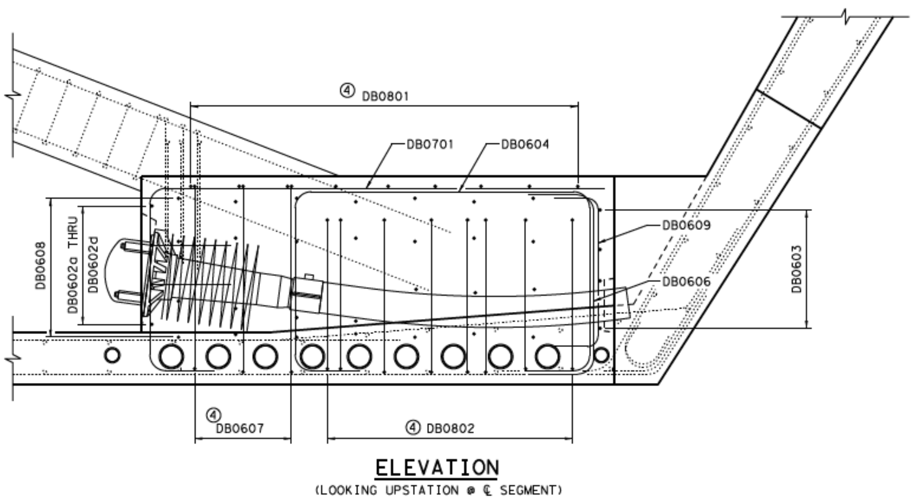


Figure 2: Delta Frame Tendon DT2 & DT3 Anchor Block Details (NB Shown, SB Similar) – Schematics by Others

These unreinforced interfaces, which must be able to transfer axial load, shear, and flexure from the box girders to the delta frames, lack a ductile means to adequately resist flexure and shear at this critical connection. The DF2 and DF3 post-tensioning tendons cross two of these interfaces. However, AASHTO LRFD §5.8.4.1 does not include post-tensioning as an acceptable means to resist interface shear. Further, literature cited in the AASHTO LRFD Commentary and other relevant literature lack experimental research to support the sole use of post-tensioning to provide ductile interface shear resistance without interface reinforcement allowed by AASHTO LRFD §5.8.4.1: single bars, multiple leg stirrups, or welded wire fabric.

Flexural moments under Service (and Strength) limit state loadings cause top fiber tensile stresses across the interface that exceed the no tension allowable limits of AASHTO LRFD §5.9.4.2.2. There is no reinforcement to arrest cracking in the pour-back, and so the current design also does not meet the requirements of AASHTO LRFD §5.7.3.4. As cracks deepen, the ability of the interfaces to resist shear decreases. The shear from each box girder (NB and SB) must transfer through these interfaces to the delta frame on its way to the support at the stays. Well anchored reinforcement must be provided across both interfaces of the pour-back to satisfy the above-referenced requirements. Without this reinforcement, brittle failure of these critical connections cannot be ruled out.

AASHTO LRFD §5.8.4 provides formulas to compute interface shear resistance. However, calculations using these formulas are not valid unless the AASHTO LRFD §5.8.4.1 and §5.8.4.4 minimum interface reinforcement requirements are satisfied. To further demonstrate the severity of the problem, Appendix A presents sample computations of shear demand and interface shear resistance using these formulas considering an assumed crack depth. The 1.97 shear demand-to-capacity ratio computed would decrease significantly, and the AASHTO LRFD requirements discussed could be satisfied, if sufficiently anchored continuous reinforcement were provided across the pour-back interfaces. The demand-to-capacity ratio could also be improved by intentional roughening. However, roughening alone would not address the lack of ductility, and it would not satisfy the AASHTO LRFD §5.8.4.1 and §5.8.4.4 minimum interface reinforcement requirements.

The unreinforced pour-back also does not meet some of the requirements of AASHTO LRFD §5.10 – Details of Reinforcement (e.g., minimum reinforcement spacing requirements, temperature and shrinkage requirements, effects of curved tendons, etc.). Referring to Figure 2 below, the pour-backs must resist radial stresses from the 2 x 31-strand tendons that deviate through this element with a 15' radius. Without reinforcement, the pour-back does not have a means to meet the requirements of AASHTO LRFD §5.10.4.3.

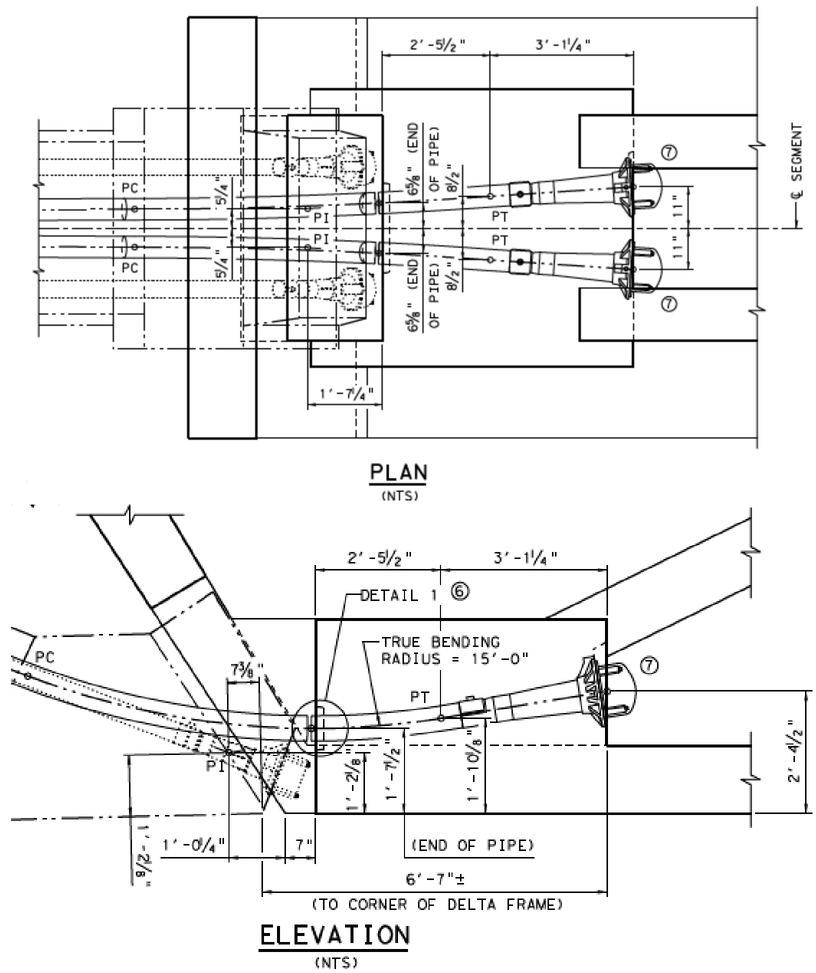


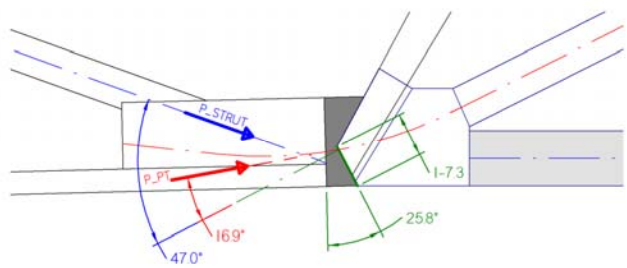
Figure 2: Delta Frame Tendon DT2 & DT3 Anchor Block Details (SB Shown, NB Similar) – Schematics by Others

The characteristics of the cast-in-place pour-back material has not yet been specified in the Current Design; therefore, the it does not meet the requirements of AASHTO LRFD §5.4.1 Also, the above-referenced FDLLC Letter shows that the delta frame surface adjacent to the cast-in-place pour-back has not been intentionally roughened to a minimum amplitude of ¼" to maximize cohesion and friction properties of the interface, as was specified for the bottom precast segment DF2 & DF3 anchor block vertical face adjacent to the cast-in-place pour-back.

4. Conclusion

The ISA has identified several deficiencies in the current Delta Frame – to – Box Girder connection design. The current design does not meet the project requirements, namely AASHTO LRFD §5.4.1; §5.8.4.1 and §5.8.4.4; §5.9.4.2.2; §5.7.3.4; and §5.10. Other findings related to the delta frame also remain unresolved, and these findings have not been addressed by changes in wind input recently observed in the updated (Rev. 2) Wind Report.

Appendix A. Hypothetical Interface Shear Calculation (Valid only if the minimum interface reinforcement requirements of AASHTO LRFD §5.8.4.1 and §5.8.4.4 are satisfied)



NB BOX GIRDER SOCKET

Delta Frame Socket Interface Shear   AASHTO LRFD § 5.8.4

$\alpha = 16.9°$ $P_{u_{STRUT}} = 991 \text{ kip} \times 2 = 1982 \text{ kip}$ $f'_c = 10{,}000 \text{ psi} = 10 \text{ ksi}$

$\beta = 47.0°$

$\mu = 0.6$ $\phi = 0.9$ $P_{u_{PT}} = 1855 \text{ kip}$ (51% f_{pu} $2 \times 31\ \phi 0.6''$)

$c = 0.075 \text{ ksi}$

$A_{cv} = 6'\text{-}2'' \times 1'\text{-}7'' = 74 \text{ in} \times 19 \text{ in} = 1406 \text{ in}^2$

$$V_u = 1855 \text{ kip} \cdot \sin 16.9° + 1982 \text{ kip} \cdot \sin 47.0°$$
$$= 539 \text{ kip} + 1450 \text{ kip} = 1989 \text{ kip}$$

$$N = 1855 \text{ kip} \cdot \cos 16.9° + 1982 \text{ kip} \cdot \cos 47.0°$$
$$= 1775 \text{ kip} + 1351 \text{ kip} = 3127 \text{ kip}$$

$$\phi V_{n_i} = \phi(cA_v + \mu N) = 0.9(0.075 \cdot 1406 \text{ in}^2 + 0.6 \cdot 3127 \text{ kip})$$
$$1783 \text{ kip} < V_u \quad \text{N.G.}$$
$$D/C = 1.12$$

Check AASHTO LRFD § 5.8.4.1 limits:

Eq 5.8.4.1-4 : $K_1 = 0.2$
; -5 : $K_2 = 0.8 \text{ ksi}$

$V_{ni} = K_1 f'_c A_{cv}$ eq. 1-4
$= 0.2(10 \text{ ksi})(1406 \text{ in}^2)$
$= 2812 \text{ kip}$

$V_{ni_{max}} = K_2 A_c$ eq. 1-5
$= 0.8 \text{ ksi} \cdot 1406 \text{ in}^2$
$= 1125 \text{ kip} \Rightarrow \phi V_{ni_{max}} = 1012 \text{ kip}$

$D/C = 1.97$

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

TECHNICAL MEMORANDUM

EXPANSION JOINT PIER SEGMENTS - UPLIFT

DOCUMENT NUMBER: TM1004

08/12/2022

Revision 0

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

12 August 2022

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

TECHNICAL MEMORANDUM

EXPANSION JOINT PIER SEGMENTS - UPLIFT

DOCUMENT NUMBER: TM1004

ORIGINATORS: Greg Glass, PE (CA) and Michael W. Smart, PE

Revision History

Revision	Date	Description
0	08/12/2022	Original Issue

Table of Contents

1. Introduction

This technical memorandum discusses a previously-reported finding of the Independent Structural Analysis (ISA) concerning uplift at the disc bearings supporting the expansion joint pier segments at piers 2N and 2S of the Corpus Christi New Harbor Bridge, cable-stay main bridge. The current design does not meet the project requirements.

2. References

The following documents are referenced in this memorandum.

1. Texas Department of Transportation (TxDOT), "Technical Provisions for US 181 Harbor Bridge Project: Comprehensive Development Agreement." ["TP"]
2. American Association of State Highway and Transportation Officials (AASHTO), "LRFD Bridge Design Specifications," 7th Edition, 2014 with 2015 Interim Revisions. ["AASHTO LRFD"]
3. "277609-NHB-PLN-MSUPER_B-01" Sheet Nos. NHB 177-184C for Expansion Joint Pier Segment Details, "277609-NHB-PLN-MSUB_B-00" Sheet Nos. 33 & 34C for Vertical Stay Details, and "277609-NHB-PLN-M13B+C-01" Sheet Nos. NHB 121-124A for Bearing Details ["Design Drawings" or "Current Design"]
4. "277609-NHB-REP-MWER-02: US181 Harbor Bridge Replacement Project: Wind Engineering Report," Revision 2, May 4, 2021 (First received by ISA Team June 7, 2022) ["Wind Report"]
5. Flatiron Dragados LLC, "FDLLC-RFI-000111: Back Span Pier Vertical Tendon," dated June 26, 2017. ["RFI 111"]
6. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 1010 dated January 8, 2021 ["ISA Phase 1 Report"]
7. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 2010 dated April 23, 2022 ["ISA Phase 2 Part 1 Report"]
8. Meeting Notes of 26 May 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["May 2022 Meeting"].
9. Meeting Notes and Presentations of 10 June 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["June 2022 Meeting"]
10. Meeting Notes and Presentations of 29 July 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["July 2022 Meeting"]

3. Background

TP §13.2.1.1 requires that the "Developer shall proportion bridge spans to avoid uplift at supports. Permanent tie-downs are prohibited." RFI 111 seeks relief from TP §13.2.1.1 by requesting acceptance of tie-downs at the backspan piers, which include transition piers 2N and 2S, using vertical tendons. This RFI assured that "The vertical tendon will connect the pier and superstructure in order to preclude any actual uplift under these conservative Factored Load conditions. The prestressing force in these vertical tendons will preload the bearings in compression to alleviate any resultant uplift."

4. Summary of Finding

In the May 2022 Meeting, the ISA Team confirmed a previously-reported finding (identified in the ISA Phase 1 Report and subsequently in the ISA Phase 2 Part 1 Report) that bearing uplift under both Service and Strength loadings occurs in the current design. In this meeting, the Developer's Lead Engineer (DLE) acknowledged that uplift occurs under Strength loadings.

With the current design, uplifted bearing(s) at piers 2N and 2S would not be able to transmit lateral loads once uplift occurs. In the May 2022 meeting, the DLE indicated that they were working with a bearing supplier to develop details that would ensure that the bearings would retain their lateral capacity given uplifted deck displacements. Additionally, the DLE has indicated that they do not intend to design the bearings to resist uplift. The DLE's actions indicate that bearing uplift will be allowed to occur, which does not meet the project requirements. Also, the ISA Team does not agree with the validity of such an approach, as briefly explained in the following paragraphs.

Following the instructions of RFI 111, it was assumed that there would be no bearing uplift. However, the ISA analysis results reveal that uplift does occur. Since the bearings as currently designed do not have uplift restraint capacity, tension reactions would shift to the tendons and non-linear behavior would result. This is inconsistent with the ISA and the analyses of others – namely the frequency domain dynamic analyses documented in the Wind Report which assumes linear superposition.

If uplift were allowed to occur at piers 2N or 2S, the box girder would lift off from at least one of its bearings, resulting in an elongation of vertical tie-down stays and a redistribution of the loads between the compressed bearing(s)/tie-down stay(s) system and the superstructure through additional axial load, shear, flexure, and torsion. Bearing uplift introduces non-linear behavior that would result in a reduction of the vertical, lateral, and torsional stiffness of the superstructure connection at piers 2N and 2S, which is not considered in the analyses. It would be inconsistent with a fundamental assumption in the Wind Report and in other analysis models that presume the structure remains within the linear domain under the design wind loadings. Appendix E of the Wind Report explicitly states, "The equivalent static wind load analysis is based on the assumption of linear elastic behaviour and results in different load cases which can be analysed to develop an envelope of stress conditions in the structure. These loads are suitable for a linear structure for both positive and negative loads."

In addition, unseating of bearings would likely lead to unacceptable bearing performance and maintenance issues.

5. Conclusion

The ISA has identified bearing uplift at both the NB and SB box girders at piers 2N and 2S, and this does not meet the project requirements, specifically TP §13.2.1.1 / RFI 111. The DLE has also acknowledged that uplift occurs under AASHTO Strength load combinations. Since bearing uplift under any limit state is not allowed, and since uplift is incompatible with the Wind Report and other structural models, calculations attempting to model and compute post-uplift non-linear behavior are not justified.

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

TECHNICAL MEMORANDUM

ERECTION LOADING

DOCUMENT NUMBER: TM1005

08/12/2022

Revision 0

Prepared For: Texas Department of Transportation

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

STATE OF TEXAS
MICHAEL W. SMART
113501
LICENSED
PROFESSIONAL ENGINEER

12 August 2022

Prepared By: SYSTRA INTERNATIONAL BRIDGE TECHNOLOGIES

9325 Sky Park Court Suite 320
San Diego, CA 92123

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

TECHNICAL MEMORANDUM

ERECTION LOADING

DOCUMENT NUMBER: TM1005

ORIGINATOR: Michael W. Smart, PE

Revision History

Revision	Date	Description
0	08/12/2022	Original Issue

Table of Contents

1. Introduction

This technical memorandum discusses a finding of the Independent Structural Analysis (ISA) concerning erection loadings, which take place during the construction of the Corpus Christi New Harbor Bridge, cable-stay main bridge. The current design does not meet the project requirements.

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-MSUPER_A-0" ["Design Drawings" or "Current Design"]
3. "277609-NHB-REP-MWER-02: US181 Harbor Bridge Replacement Project: Wind Engineering Report," Revision 2, May 4, 2021 (First received by ISA Team June 7, 2022) ["Wind Report"]
4. "277609-NHB-MAN-MEM-01_01" ["Erection Manual"]
5. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 1010 dated January 8, 2021 ["ISA Phase 1 Report"]
6. Independent Structural Analysis for the Corpus Christi New Harbor Bridge Project, Document Number: 2010 dated April 23, 2022 ["ISA Phase 2 Part 1 Report"]
7. Meeting Notes of 26 May 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["May 2022 Meeting"]
8. Meeting Notes and Presentations of 10 June 2022 meeting in Austin, TX between TxDOT, FDLLC, HNTB, ARUP-CFC, and IBT ["June 2022 Meeting"]

3. Background

AASHTO LRFD §5.14.2.3.1 requires that the partially built permanent structure shall be checked during construction with load combinations specified in Section 3. The ISA team has previously identified findings related to construction loadings, which have been presented in the ISA Phase 1 Report and the ISA Phase 2 Part 1 Report. The issues identified previously were related to the Service limit state requirements of AASHTO LRFD and to constructability. These findings have yet to be addressed by the Developer, and they have not been addressed by the change in wind input recently observed in the Rev. 2 Wind Report.

The revised Wind Report (Rev. 2) received June 7th, 2022 now provides more critical wind load cases during construction. The memorandum focuses on a Strength limit state finding that relates to the partially built structure subject to a design wind event during construction.

4. Summary of Finding

The current design of the bottom slab of the back span superstructure has insufficient post-tensioning to provide the flexural resistance necessary to withstand positive bending (tension at the bottom) caused by wind and other effects at a certain stage of construction. The state of the structure considered is shown in Figure 1 below. The loading considered occurs after the first-stage stressing of the Stay 12 pair of stays, just before the superstructure reaches the back span piers 1N and 1S.

The critical sections are located at the beginning of NB and SB segments 32. These segments are the first ones erected after the temporary bent has been connected to the deck.

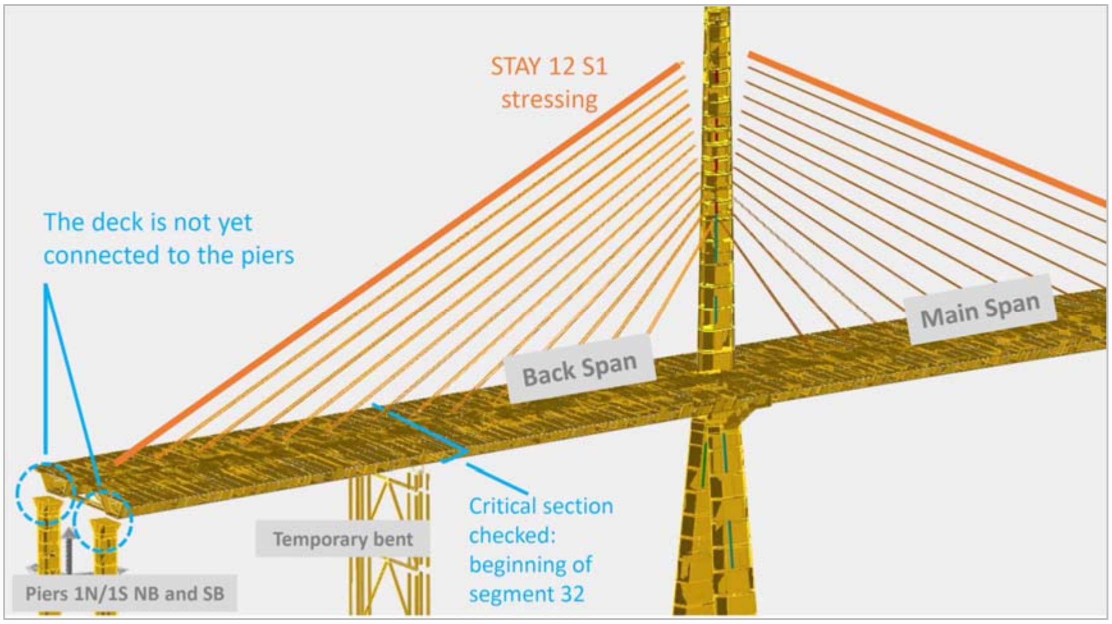


Figure 1: Stage 12 S1 Stressing Showing Erection Phase and Critical Section Investigated

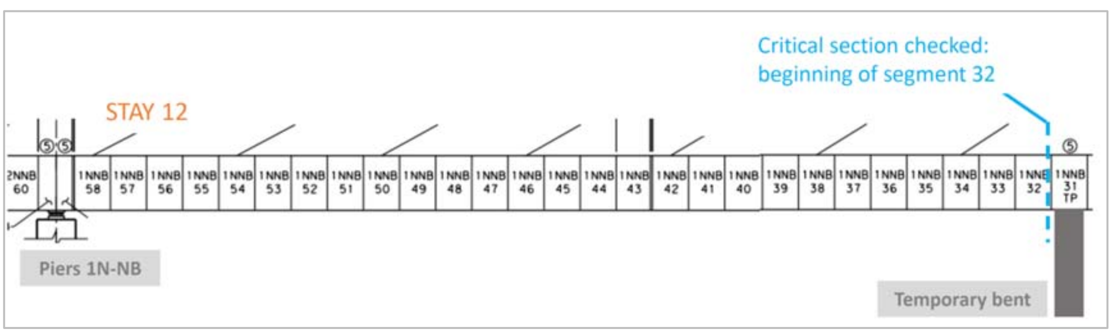


Figure 2: Elevation View Showing Critical Section Investigated (NB shown – SB similar)

At this phase of construction, these segments experience large positive bending moment (tension at the bottom) due to the first stage stressing and due to the factored wind demand for that configuration, identified as BLWTL's Wind Case 2 (originally named "Construction Case #2, free cantilever, before arrival to the back span pier (temporary pier still active), for West wind direction") with only five post-tensioning bars located in the bottom slab to provide resistance.

The governing combination is Strength III with the minimum required load factor of 1.25 applied to wind during construction, with either 1.25 or 0.90 applied to DC, EL, and CR/SH, and 1.00 applied to secondary post-tensioning (AASHTO LRFD §3.4.2.1), so the ISA considered the following two combinations:

Strength III-A: 1.25 DC + 1.25 EL + 1.00 PS2 + 1.25 CR/SH + 1.25 WS

Strength III-B: 0.90 DC + 0.90 EL + 1.00 PS2 + 0.90 CR/SH + 1.25 WS

Where:

- DC: Dead load of the structure and of the erection equipment
- EL: Stay load
- CR/SH: Creep and shrinkage
- WS: Wind on the structure for BLWTL's Wind Case 2

Demands for individual load effects, the Strength III-A and Strength III-B load combinations, and capacity interaction plots are presented in Appendix A. Table 1 below presents a summary of demand from these load combinations along with corresponding demand-to-capacity (D/C) ratios.

Table 1: Demand and D/C Ratios for Segments 32 NB/SB

	Deck SB/NB	FX (kips)	MY (kip-ft)	MZ (kip-ft)	D/C
Strength III-A	**SB**	-10,548	-43,744	136,277	**2.16**
Strength III-B	**SB**	-7,146	-32,070	122,071	**2.55**
Strength III-A	**NB**	-12,656	69,409	126,006	**1.90**
Strength III-B	**NB**	-8,698	49,205	115,585	**2.37**

Where:

- FX: axial demand in the deck, negative in compression
- MY: transverse bending moment
- MZ: longitudinal bending moment (positive for tension at the bottom of the section)

Appendix B presents similar computations using available demands provided in the Erection Manual by others. D/C ratios using these demands ranged from 1.74 – 2.20.

5. Conclusion

The ISA has identified insufficient post-tensioning in the bottom slab of sections in the back span superstructure over 121' due to wind during construction. Flexural demand exceeds capacity at this location subject to Strength limit state loadings. The applicable requirements are AASHTO LRFD §1.3.2.1, §5.14.2.3.1, §3.4.2.1, and §5.7.3.2. Mitigation measures to address this finding were not documented in the Erection Manual nor in any of the other Current Design documents.

Other findings related to construction loadings caused by excessive torsional loadings considering shear lag 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

The findings related to construction presented previously have not yet been addressed by the Developer.

Appendix A. Calculations Considering ISA Computed Loadings

A.1 Section Investigated

The NB/SB section investigated is at the beginning of segment 32, which is located at station 1085+78.94 (at x=-1135 ft, where x = 0 at midspan of the main span). This is the most critical section, but the problem extends over 121 ft from stations 1084+57 to 1085+78. The cross-section for the critical section with its post-tensioning are as follows:

Table 2: Post-Tensioning Bars and Tendons

Position: top/bottom	Cable/Bar name	Nb	size SB	size NB
top	BT5	2	Tendon 25 strands 0.6''	Tendon 25 strands 0.6''
top	BT8	2	Tendon 27 strands 0.6''	Tendon 27 strands 0.6''
top	R4	2	PT bar 1.38 in diameter	PT bar 1.75 in diameter
top	R5	2	PT bar 1.75 in diameter	PT bar 1.75 in diameter
top	R6	2	PT bar 1.75 in diameter	PT bar 1.75 in diameter
top	R7	2	PT bar 1.75 in diameter	PT bar 1.75 in diameter
top	R8	2	PT bar 1.75 in diameter	PT bar 1.75 in diameter
top	R9	2	PT bar 1.75 in diameter	PT bar 1.75 in diameter
top	R10	2	PT bar 1.38 in diameter	PT bar 1.25 in diameter
top	R11	2	PT bar 1.38 in diameter	PT bar 1.25 in diameter
bottom	R1	1	PT bar 1.38 in diameter	PT bar 1.25 in diameter
bottom	R2	2	PT bar 1.38 in diameter	PT bar 1.25 in diameter
bottom	R3	2	PT bar 1.38 in diameter	PT bar 1.25 in diameter

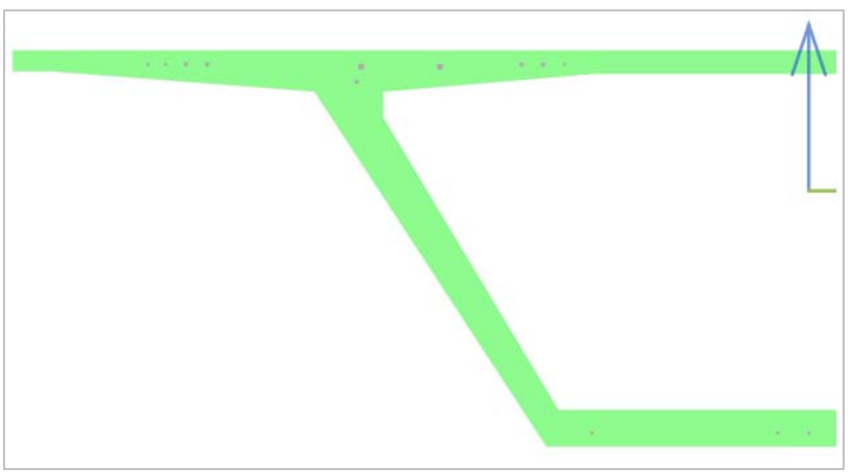


Figure 3: SB Section Investigated Showing PT Bars and Tendons

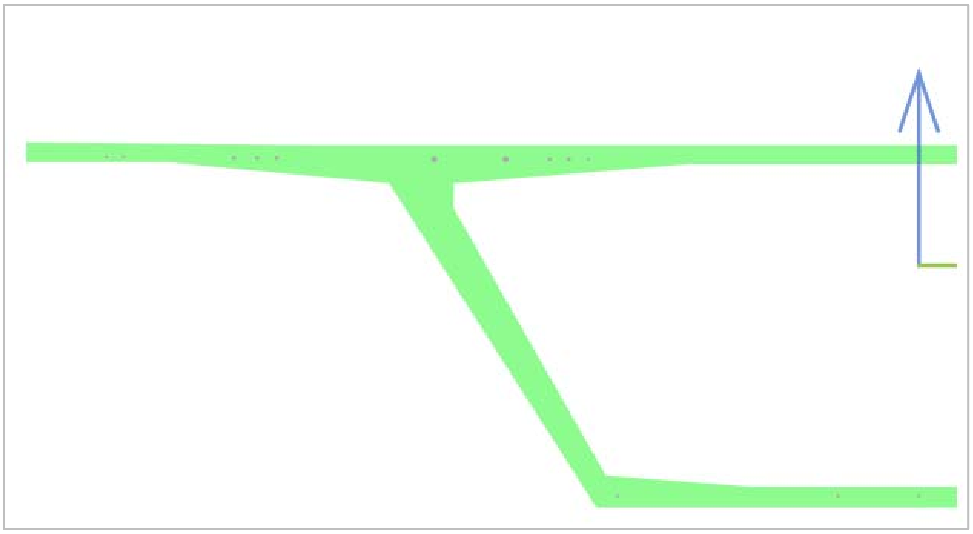


Figure 4: NB Section Investigated Showing PT Bars and Tendons

A.2 Load Combination - Demand

The governing combination is Strength III:

Strength III-A: 1.25 DC+ 1.25 EL + 1.00 PS2 + 1.25 CR/SH + 1.25 WS

Strength III-B: 0.90 DC+ 0.90 EL + 1.00 PS2 + 0.90 CR/SH + 1.25 WS

Unfactored demands are presented below along with the factored combinations Strength III-A and Strength III-B. The stage total demand (unfactored), after Stay 12 stage 1 stressing is also provided for information. DC demand includes the sum of DC-structure (self-weight) and DC-erection (all erection loads).

SB:

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	Comb A	Comb B
Stage total demand	-16,870	-36,821	60,462		
DC+EL	-9,079	-44,884	42,430	**1.25**	**0.90**
PS1	-7,432	11,656	21,874	**0.00**	**0.00**
PS2	282	-15,124	-2,001	**1.00**	**1.00**
CRSH	-641	11,531	-1,840	**1.25**	**0.90**
WS (governing case)	1,056	10,457	70,033	**1.25**	**1.25**

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	D/C
Strength III-A	**-10,548**	**-43,744**	**136,277**	**2.16**
Strength III-B	**-7,146**	**-32,070**	**122,071**	**2.55**

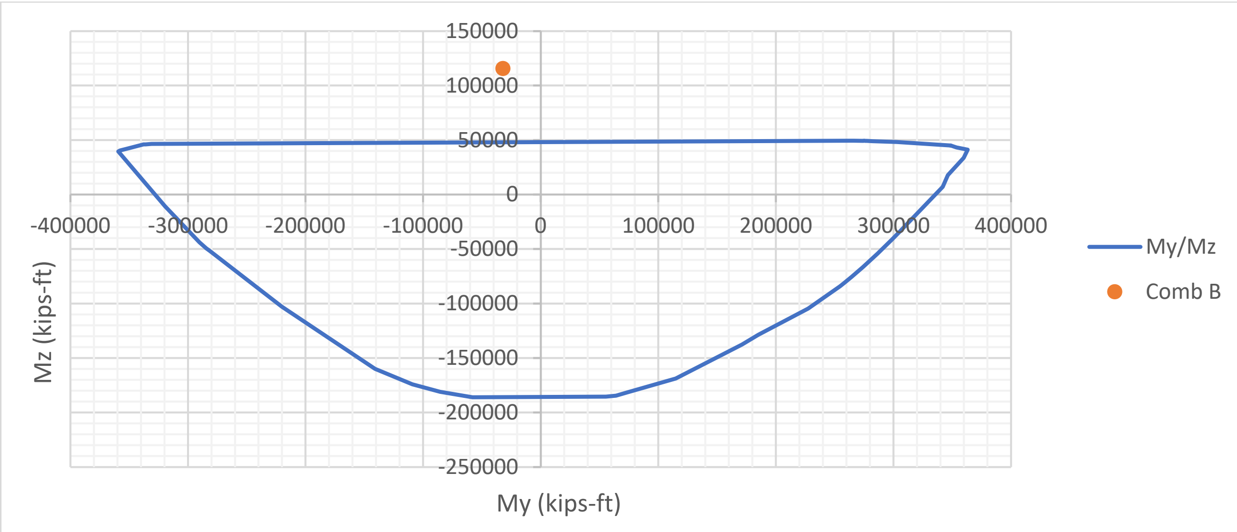


Figure 5: SB Interaction (Fx-My-Mz) Diagram for Governing Case (Fx=-7146 kips)

NB:

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	Comb A	Comb B
Stage total demand	-18,398	58,439	50,137		
DC+EL	-10,828	72,917	29,524	**1.25**	**0.90**
PS1	-7,295	-16,840	21,764	**0.00**	**0.00**
PS2	205	17,553	-1,401	**1.00**	**1.00**
CRSH	-480	-15,191	250	**1.25**	**0.90**
WS (governing case)	1,019	-16,242	72,152	**1.25**	**1.25**

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	D/C
Strength III-A	**-12,656**	**69,409**	**126,006**	**1.90**
Strength III-B	**-8,698**	**49,205**	**115,585**	**2.37**

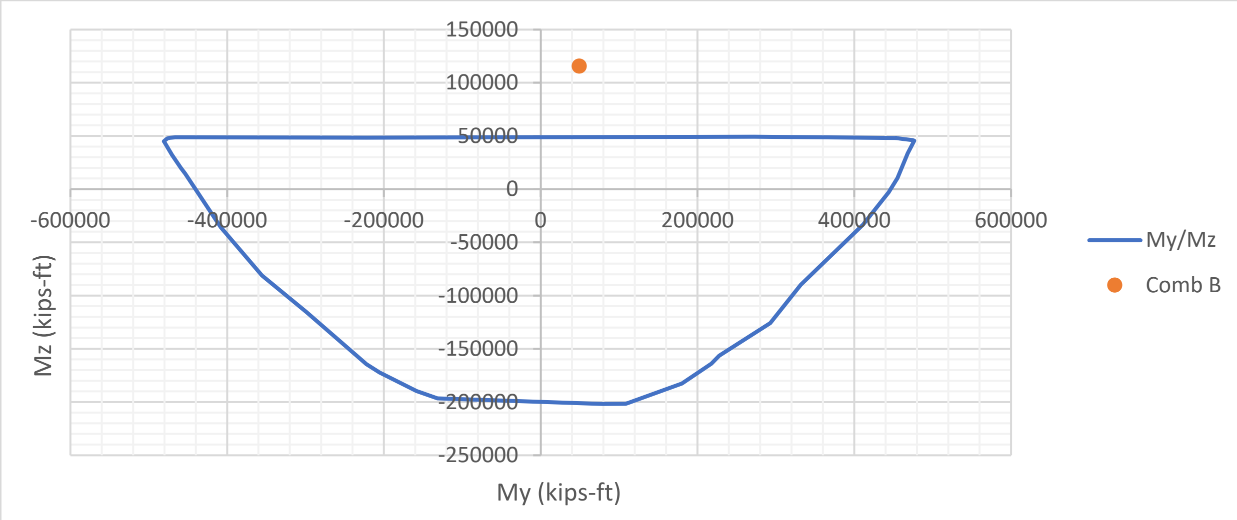


Figure 6: NB Interaction (Fx-My-Mz) Diagram for Governing Case (Fx=-8698 kips)

Where:

- FX: axial demand in the deck, negative in compression
- MY: transverse bending moment
- MZ: longitudinal bending moment, positive for sagging
- DC: Dead load of the structure and of the erection equipment
- EL: Stay load
- PS1: Primary PT effect, excluded from external loads
- PS2: Secondary PT effect
- CR/SH: Creep and shrinkage
- WS: Wind on the structure for BLWTL's Wind Case 2

Appendix B. Calculations Considering Erection Manual Loadings

B.1 Erection Manual Demand

The intent of this section is to investigate the same section as the one presented in Appendix A, considering the demand that is provided in the Erection Manual.

The construction demand is extracted from the Erection Manual plots for the stage under consideration (Stage C12_Stay 12 S1- Sheet 104 of 171).

This demand excludes primary PT, and it does not provide the breakdown between DC, EL, PS2, and CR/SH. Therefore, the Strength combination III cannot be calculated directly from the plots that are provided. Instead, the ISA used selected information from its analysis, so that it could apply load factors and calculate total demand consistently. Figures 7, 8, and 9 below shows axial load, longitudinal bending moment, and transverse bending moment demand for the NB and SB box girders from the Erection Manual. In the tabulation for SB, Figure 10 below shows the demand obtained from the Erection Manual along with ISA-computed primary PT and overall demand at this stage of construction. It also shows how total demand exceeds capacity with a biaxial interaction diagram (Fx-My-Mz). Figure 11 shows the same for NB.

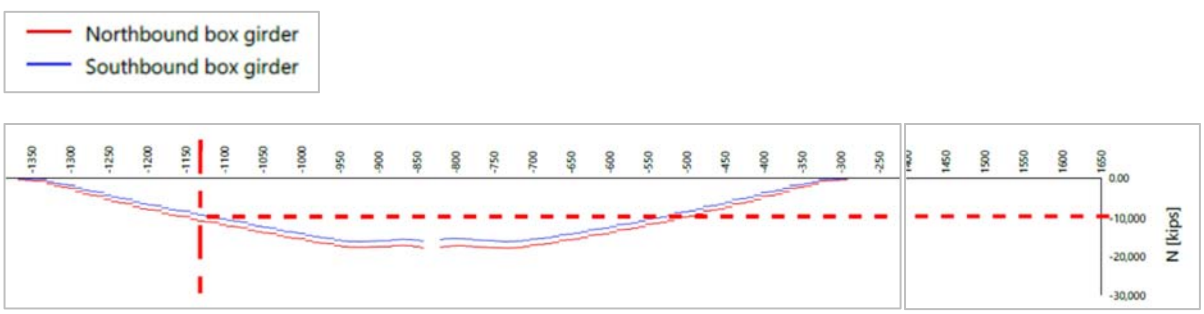


Figure 7: Erection Manual Axial Demand in NB/SB Decks for Stage Stay_12 S1

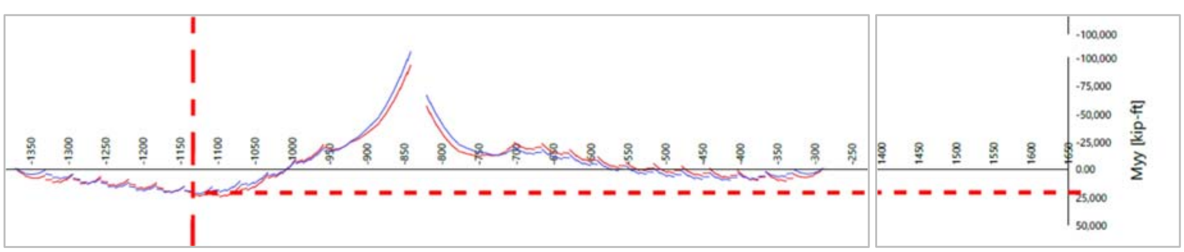


Figure 8: Erection Manual Longitudinal Bending Demand in NB/SB Decks for Stage Stay_12 S1

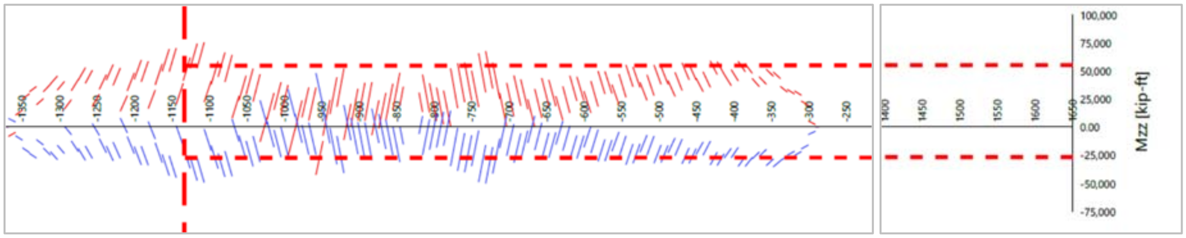


Figure 9: Erection Manual Transverse Bending Demand in NB/SB Decks for Stage Stay_12 S1

SB:

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	γ_A	γ_B
Stage Dmd_{ISA}	-16,870	-36,821	60,462		
Stage $Dmd_{Erect.Manual\ w/o\ PS1}$	-9,900	-28,000	21,000		
$PS1_{ISA}$	-7,432	11,656	21,874		
Stage $Dmd_{Erect.Manual\ w/o\ PS1}$+$PS1_{ISA}$-Stage Dmd_{ISA}	**-462**	**20,477**	**-17,589**	**1.25**	**0.9**

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	D/C
Strength III-A Dmd_{ISA}	-10,548	-43,744	136,277	**2.16**
Strength III-B Dmd_{ISA}	-7,146	-32,070	122,071	**2.55**
Strength III-A $Dmd_{Erection\ Manual}$	**-11,126**	**-18,148**	**114,291**	**1.74**
Strength III-B $Dmd_{Erection\ Manual}$	**-7,562**	**-13,641**	**106,241**	**2.13**

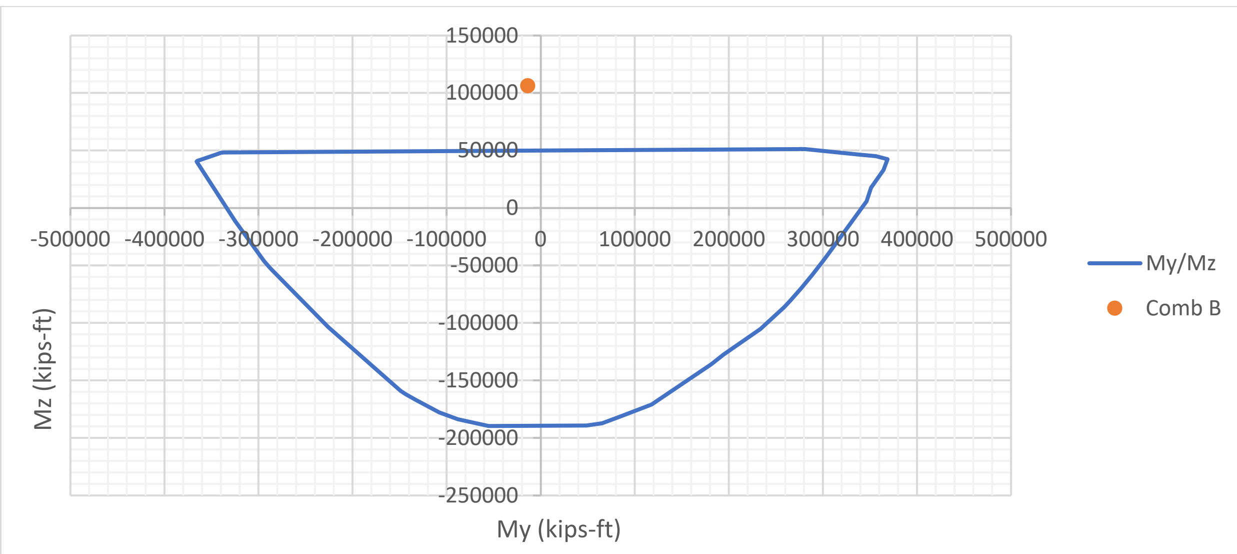


Figure 10: SB Interaction (Fx-My-Mz) Diagram for Governing Case (Fx = -7562 kips) with Demand from Erection Manual

NB:

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	γ_A	γ_B
Stage Dmd_{ISA}	-18,398	58,439	50,137		
Stage $Dmd_{Erect.Manual\ w/o\ PS1}$	-11,500	55,000	23,000		
$PS1_{ISA}$	-7,295	-16,840	21,764		
Stage $Dmd_{Erect.Manual\ w/o\ PS1}$+$PS1_{ISA}$-Stage Dmd_{ISA}	**-397**	**-20,279**	**-5,373**	**1.25**	**0.9**

	FX (kips)	MY (kip-ft)	MZ (kip-ft)	D/C
Strength III-A Dmd_{ISA}	-12,656	69,409	126,006	**1.90**
Strength III-B Dmd_{ISA}	-8,698	49,205	115,585	**2.37**
Strength III-A $Dmd_{Erection\ Manual}$	**-13,152**	**44,060**	**119,290**	**1.75**
Strength III-B $Dmd_{Erection\ Manual}$	**-9,055**	**30,953**	**110,750**	**2.20**

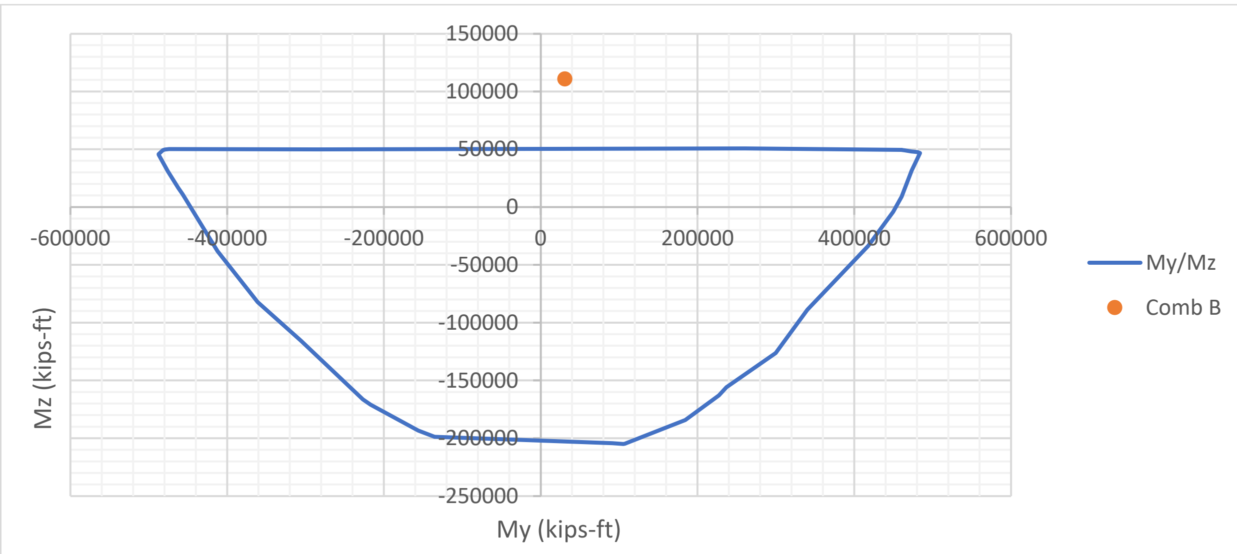


Figure 11: NB Interaction (Fx-My-Mz) Diagram for Governing Case (Fx=-9055 kips) with Demand from Erection Manual