Texas Department of Transportation

1701 S. PADRE ISLAND DRIVE, CORPUS CHRISTI, TEXAS 78416 | 361.808.2660 | WWW.TXDOT.GOV

April 29, 2022

Mr. Keith Armstrong
Project Manager
Flatiron/Dragados, LLC
500 N. Shoreline Blvd., Suite 500
Corpus Christi, Texas 78401

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

Dear Mr. Armstrong:

This letter is sent pursuant to Section 6.8 of the Comprehensive Development Agreement ("CDA"), dated September 28, 2015, between the Texas Department of Transportation ("TxDOT") and Flatiron/Dragados, LLC ("FDLLC") and constitutes a Notice of Nonconforming Work. Capitalized terms not defined in this Notice shall have the meanings given to them in the CDA.

TxDOT retained International Bridge Technologies ("IBT") to provide an independent review of ARUP-CFC's design of the New Harbor Bridge. Attached to this letter as Exhibit A is a report prepared by IBT dated April 23, 2022 (the "IBT Report") detailing its findings regarding FDLLC's design. The IBT Report identifies a number of instances of Nonconforming Work[1] 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. Attached to this letter as Exhibit B is a table summarizing certain findings by IBT and corresponding provisions of the Contract Documents that FDLLC has breached. The table is intended as a high-level summary and FDLLC should refer to the complete IBT Report for a full description of IBT's findings and the numerous instances of Nonconforming Work.

Each of the findings summarized in Exhibit B and the other findings in the IBT Report not listed in Exhibit B identify Nonconforming Work, which is hereby rejected by TxDOT. Pursuant to Section 6.8.1 of the CDA, FDLLC is required to remove and replace all of the Nonconforming Work so as to conform to the requirements of the Contract Documents at FDLLC's cost and without any adjustment to the Price or any Completion Deadline. Further, FDLLC shall promptly take all action necessary to prevent similar Nonconforming Work from occurring in the future. If FDLLC fails to correct all the Nonconforming Work within ten days of receipt of this notice, or if such Nonconforming Work cannot be corrected within ten days and FDLLC fails or refuses to: (a) provide to TxDOT a schedule

[1] "Nonconforming Work" is defined in Exhibit 1 the CDA to mean "Work that does not conform to the requirements of the Contract Documents, the Governmental Approvals, applicable Law or the Design Documents."

acceptable to TxDOT for correcting all such Nonconforming Work within such ten day period, (b) commence such corrective work within such ten-day period, and (c) thereafter diligently prosecute such correction in accordance with such approved schedule to completion, then TxDOT may cause the Nonconforming Work to be remedied or removed and replaced and may deduct the cost of doing so from any moneys due or to become due FDLLC or obtain reimbursement from FDLLC for such cost. In addition, any failure or refusal to follow these contractual requirements, including the timely removal and replacement of the Nonconforming Work will constitute a Developer Default pursuant to CDA Section 16.1.1(c). TxDOT is continuing to review the findings in the IBT Report and reserves the right to identify additional breaches of the CDA by FDLLC arising out of the matters described therein.

We look forward to your prompt response.

Sincerely,

DocuSigned by:
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Valente Olivarez, Jr., P.E.
Corpus Christi District Engineer
Texas Department of Transportation

Attachments: Exhibit A “Independent Structural Analysis for the Corpus Christi New Harbor Bridge”
Exhibit B “Summary of Selected IBT Findings and CDA Provisions”

cc: Hugo Fontirroig, Project Executive, FDLLC
Justo Molina, Project Executive, FDLLC
Kurt Knebel, Vice President & Texas District Manager, Flatiron Constructors, Inc.
Jaime Hurtado Cola, Legal Counsel, Dragados, SA
Jose Luis Mendez, President – Dragados USA
Javier Sevilla, Flatiron
Marc D. Williams, P.E., Executive Director, TxDOT
Joseph Briones, P.E., Corpus Christi District Project Manager, TxDOT
John Becker, P.E., HNTB

Exhibit A

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Legacy Contract No. 88-0SDP5002 PS 10781

Phase 2 Part 1 REPORT – CABLE-STAYED MAIN BRIDGE

SUMMARY DOCUMENT

DOCUMENT NUMBER: 2010

04/23/2022

Revision 1

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

23 April 2022

INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

Phase 2 Part 1 REPORT - CABLE-STAYED MAIN BRIDGE

SUMMARY DOCUMENT

DOCUMENT NUMBER: 2010

Revision History

Revision	Date	Description
0	4/19/2022	PHASE 2 PART 1 REPORT
1	4/23/2022	CLARIFYING EDITS AND TYPOS CORRECTED

Table of Contents

EXECUTIVE SUMMARY

This document presents a summary of the structural design checking of the cable-stayed main bridge, performed by the Independent Structural Analysis (ISA) team for the new Corpus Christi Harbor Bridge (CCHB), prepared for the Texas Department of Transportation (TxDOT).

The ISA scope of work has been divided into two phases. Phase 1 considered the structural design as submitted by the original main bridge designers, and Phase 2 considers the updated structural design as submitted by the new main bridge designers. For a summary of Phase 1 efforts and more context, refer to the 100% Final Report – Cable-Stayed Bridge Summary Document – Rev. 0, 08 January 2021, Doc. # 1010 (ISA Phase 1 Report).

The report herein summarizes Part 1 of Phase 2, which covers the ISA team's efforts to-date checking the updated structural design as submitted by the Design-Build team's new main bridge designers. The ISA checking effort is still in progress. However, it is understood that the Design-Build team considers its updated design complete, and main bridge construction operations have resumed. Due to project schedule urgency and the importance of ISA findings to date, it was decided to present Phase 2 of the ISA into two parts, so that the Design-Build team may be made aware of ISA main bridge findings concerning its updated design as soon as possible in advance of final completion of the ISA Phase 2 work:

- Part 1 (this report) focuses on findings to-date, and
- Part 2 (to be submitted at a future date) will include items not currently complete, and will be more comprehensive, including discussion about items found to meet project requirements.

This independent checking effort included the work of structural and geotechnical engineers to review, assess, analyze, and check the design in accordance with the project requirements and documents. Design checks were carried out independently without direct exchange of information or meetings with the Design-Build team. Only documents provided by TxDOT were used as the basis for ISA calculations and assessments. In most cases, sufficient input was available, so that independent calculations could be performed by the ISA. Where sufficient input was not available (e.g. revised and updated dynamic wind input), the ISA team relied on its independent calculations performed for Phase 1, or it made reasonable assumptions. When possible and applicable, comparisons have been made between ISA results and available results provided in the documents submitted by the Design-Build team.

Main bridge structural elements were reviewed to verify their adequacy as required by the project criteria, design codes, and other references (project requirements), and results are summarized herein.

Not all necessary inputs were available to the ISA team as of this writing. However, with the information currently available, the ISA team has identified 35 unresolved findings to date concerning critical parts of the CCHB main bridge design that warrant action. There are 7 findings from Phase 1 that have been resolved by the updates to the design observed in Phase 2. The unresolved and resolved findings are summarized in Section 2 of this report, and are discussed in more detail in Section 4.

1. Introduction

This Phase 2 Part 1 Report summarizes calculations performed by the Independent Structural Analysis (ISA) team for the new Corpus Christi Harbor Bridge (CCHB) Project. For this document, the scope of the review includes the cable-stayed main bridge crossing the harbor, as shown in the figure below.

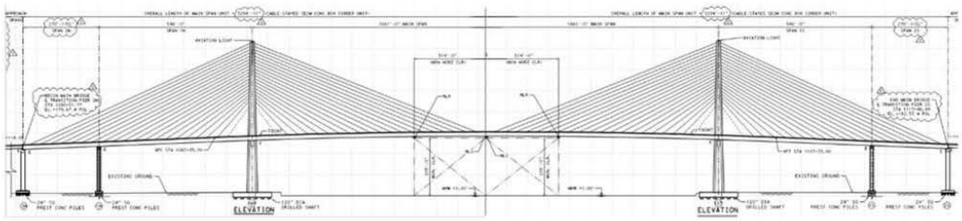


Figure 1: Cable-Stayed Main Bridge – Schematic Provided by Others

The ISA team reviewed the technical information supplied by the Texas Department of Transportation (TxDOT) and performed an independent design review of the structural components of the main cable-stayed bridge. The level of effort was consistent with industry practice for a bridge of this size and complexity. The ISA calculations include the engineering evaluations necessary to determine whether the structures satisfy or do not satisfy the project requirements.

This document includes a summary of the ISA team's design review, which identifies findings describing structural components that were found to be deficient (i.e. did not satisfy the project structural design requirements). This report also includes additional details to support the ISA team's approach and overview of results.

1.1. Document Format

The document is divided into the following sections:

Section 1 – Introduction (this section)

Section 2 – Summary of Findings

Section 3 – Reference Material

Section 4 – Findings

1.2. Exclusions and Limitations

The Independent Structural Analysis (ISA) was performed using the available project documents as supplied by the Texas Department of Transportation (TxDOT). The work was performed independently from the Design-Build team or other parties involved in the CCHB project (other than TxDOT).

As part of that separation, the ISA did not attend meetings or have other communications about this project with the Design-Build team to date. Therefore, the findings and observations in this report are strictly limited to the documented information received by the ISA team from TxDOT. The scope of the ISA work excludes items not included in the information packages received from TxDOT.

Durability analyses are excluded from the scope of the ISA. In particular, the cover requirements provided by the Design-Build team on the general notes of the design drawings are presumably based on a corrosion protection plan. Without expressing an opinion on the suitability of these values, we have considered them to be accepted by others as valid for this project, including values that may be less stringent than those specified by the TxDOT requirements. Similarly, the adequacy of the locations within the structure where epoxy coated reinforcement has been required / provided have not been reviewed by the ISA Team.

Detailed checking of the adequacy of bar shapes, lap / splice lengths, hook dimensions and compatibility of bar lengths with cover requirements have been excluded from this review. In some cases the design drawings provided sufficient detail to determine such information, and in some cases the ISA was able to provide opinions in this regard; however, such checks should not be considered exhaustive.

Review of the Project Specifications provided by the Design-Build team has been excluded from the scope of this review.

The ISA did not include independent development of the Geotechnical Report, the Redundancy Report, the Erection Manual, or the Wind Report. Findings related to these documents have been raised; however, ISA reviews of these documents should not be considered exhaustive or independently verified. Information from these reports was relied upon and used as input for ISA calculations.

1.3. Definitions

Commonly used terms in this document are summarized below.

Bearings – Support elements between the superstructure span and the supporting pier, which can be either a steel laminated elastomeric bearing, or a mechanical bearing with capabilities to resist lateral loads and uplift loads, if required and designed accordingly.

Cable-Stayed Bridge – Refers to the structure between Piers 2N and 2S, including foundations, piers, towers, bearings, deck and stay cables (including vertical tie-downs). The part of the deck located between the main bridge Towers 1NT and 1ST is referred to as the main span or the middle span. The part of the deck located between Piers 2N and Tower 1NT and between Piers 2S and Tower 1ST is referred to as the back span or the side span or the end span.

Cable Stays (or Stay Cables) – The main tension elements (MTE) that support the main bridge superstructure between towers and piers. In the original design, the typical inclined stay cables passed through the upper towers using saddles and connected to anchors in the delta frames that actively support the superstructure between the substructure elements. However, in the updated design the inclined stay cables supporting the superstructure have been anchored at the upper towers and are no longer continuous through the towers. The stay system typically includes anchorages, strands, sheathing, anti-vandalism and corrosion protection materials and devices, etc. The vertical tension tie-downs at Piers 1N, 1S, 2N, and 2S are also stay cables.

Demand-to-Capacity Ratio – A commonly used term to describe the relationship of the loads on a structural component (Demand) compared to its structural resistance (Capacity) as defined by the applicable codes. This is often referenced as a D/C ratio. A value of 1.0 or lower signifies a structural component at capacity or with reserve capacity, and a value greater than 1.0 signifies a structural element with loading exceeding its designated capacity and designated as deficient per the applicable requirement. In some cases, the inverse of the D/C ratio, the "capacity-to-demand" ratio, is used instead. For C/D, a value > 1.0 is considered acceptable.

Column – The vertical compression member connecting the bearings at the top of pier down to the foundation cap.

Delta Frame – Precast structural elements connecting the NB and SB box girders comprised of a stay anchor block, two diagonal struts, a bottom strut, and a vertical element connected to the stay anchor block that provides a brace near the middle of the bottom strut. Delta frames are typically spaced longitudinally at four-segment intervals.

Drilled Shafts (also referred to as "shafts") – 8ft or 10ft diameter, cast in place vertical reinforced concrete foundation elements supporting the foundation cap of the Towers 1NT and 1ST.

Driven Piles –Precast prestressed 24"x24" square vertical piles installed by impact hammering into the soil and supporting the foundation of Piers 2N, 2S, 1N, 1S of the cable stayed bridge as well as the foundations of the approach piers. Driven piles are also present in addition to the drilled shafts at the foundation for Tower 1NT.

Expansion Joints – Mechanical elements provided at Pier 2N and 2S to maintain the continuity of the roadway while allowing for the expected displacement and rotation of the main bridge superstructure relative to adjacent approach superstructures.

Pile Cap (also referred to as "foundation cap", "cap", or "footing") – The structural element connecting the drilled shafts and/or the driven piles to the pier or tower legs under consideration.

Substructure – Foundations, piers, tower elements, and bearings.

Superstructure (also referred to as "deck")– Box girders, cast-in-place central median slab, delta-frame elements, and the stay cables.

Tie-Down – Vertical tension element provided at the interface between the substructure and the superstructure to counteract the expected vertical uplift forces and prevent the relative vertical displacements between the superstructure and Piers 1N, 1S, 2N, and 2S. These elements are stay elements, given their expected behavior.

Tower (also referred to as "pylon") – Vertical compression member supporting the superstructure at 1NT and 1ST by transferring the horizontal and vertical components of the forces in the stay cables to the foundation.

Tower Table (also referred to as "nodal zone") – Region connecting the superstructure to the tower at 1NT and 1ST.

2. Summary of Findings

The 2nd, 3rd, and 4th columns of the list below and the footnotes are reproduced from the Phase 1 ISA Report based on the original cable-stayed main bridge design. For consistency, the items identified have been listed in the same order as they were presented in the Phase 1 ISA Report, and they have been given a unique ID number in the 1st column in the table below. The 5th column of this table has been added to cite applicable requirements and provide notes on the revised design submitted to TxDOT by the Design-Build team. For simplicity and clarity and since both must be addressed by the Design-Build team regardless of severity, both findings and observations have now been combined into a single list.

Seven of the findings raised in the Phase 1 ISA report have been addressed by the updated design, and some of the Phase 1 findings have been partially resolved. These findings remain in the list below for consistency, and the notes indicate if issues have been resolved or partially resolved by the updated design. In the 5th column, issues identified in Phase 1 Findings 1 – 36 that were found to remain deficient are highlighted yellow, if resolved or partially resolved they are highlighted green, and if discretionary (at TxDOT's discretion) they are highlighted grey. For the Finding ID in the 1st column, a yellow highlight indicates that the finding may be considered resolved, and a green highlight indicates that a deficiency or deficiencies remain. Findings 37 and above, are new findings that have arisen from the updated design that were not identified in the Phase 1 ISA report. The 35 current findings exclude the 7 Phase 1 findings that have been addressed by the revised design.

ID	Location	Phase 1 Report Section	Phase 1 Project Criteria Deficiency	Applicable Requirements and Notes
01	Cable-Stayed Bridge, General	§3.2 Redundancy Report	The project technical provisions (13.2.1.3) include requirements to evaluate the cable-stayed bridge for redundancy. The delta frame and tower table struts should be included as part of this evaluation, but was excluded from the Redundancy Report. The ISA has identified cases where failure in the delta frame could result in subsequent failures of other elements.	Technical Provisions § 13.2.1.3 The Redundancy Report has been updated (current version is MRDND Rev01). Some of the issues raised in the ISA Phase 1 report have been addressed. However, findings, questions, and comments related to the current updated Redundancy Report remain.
02	Cable-Stayed Bridge, General	§4 Construction Sequence	The AASHTO Code requires that a predefined construction sequence be included as part of the final design documents. This documentation is missing, which will likely have an impact on the ISA results. See Note 1.	AASHTO LRFD § 2.5.3 The Design-Build team has provided an erection manual, so the issue identified in Phase 1 has been resolved with the revised design, and Note 1 below is no longer applicable. The ISA has identified other findings related to the current construction sequence documents (See Findings 37 and 41 below).

ID	Location	Phase 1 Report Section	Phase 1 Project Criteria Deficiency	Applicable Requirements and Notes
03	Cable-Stayed Bridge, General	§4.1.2 Wind Load Report	The Wind Report is specifically calibrated to the original engineer's models. Also, critical construction staging cases appear to be missing from the Report. See Note 2 for additional comments.	Technical Provisions §13.2.1.7. The Wind Report has been updated (the current version is MWER Rev01). Additional construction load cases were added to the revised report, and the original issue is considered resolved. However, other discrepancies exist related to the Wind Report and the updated design. This is addressed in Finding 39 below.
04	Main Tower Foundations	§5.1 Cable-Stayed Bridge Geotechnical Evaluation, Supplementary driven piles at 1NT	Based on a detailed non-linear analysis using FLAC3D software, it was shown that the driven piles provide very little additional support to the primary drilled shafts, and will be disregarded for the ISA design checks.	Supplemental piles are no longer considered according to current design documents.
05	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT and 1ST axial capacity.	Multiple drilled shafts do not meet the project requirements for geotechnical axial load resistance and structural flexural resistance. At foundation 1NT, 16 of the 20 drilled shafts are deficient for service and strength loads. At foundation 1ST, 12 of 20 drilled shafts are deficient under service loads and 10 of 20 are deficient under strength loads. See Note 3 for additional assessment of the drilled shaft capacity.	Technical Provisions § 13.2.1.14 TxDOT BDM Ch 2 §1 TxDOT Geotechnical Manual Ch 5 §2, Ch 5 §3 AASHTO LRFD §10.5.5.2.4
06	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT axial capacity.	The use of a 1.33 "overstress" allowance factor is not supported by the project technical requirements.	Reportedly, this issue has been resolved, and it has been agreed by all parties that the overstress allowance considered by the original EoR does not apply. The main body of the updated Geotechnical Report no longer references a 33% stress allowance. However, the appendix of the Geotechnical Report still does. Also, the drilled shaft drawings in package M02 Rev02 still mention a 33% stress allowance in the notes and a 1.33 factor is still included in the capacities reported on these drawings.
07	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT and 1ST, structural capacity	One drilled shaft at each Tower 1NT and 1ST does not meet the project requirements for load resistance. This occurs at the corner drilled shaft with uplift, and is due to shallow termination of longitudinal reinforcement.	AASHTO LRFD §5.7.4.5

ID	Location	Phase 1 Report Section	Phase 1 Project Criteria Deficiency	Applicable Requirements and Notes
08	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT and 1ST, structural detailing	Multiple locations have deficient reinforcement detailing issues. Many occur at the shaft in tension, but all shafts are deficient in meeting the minimum longitudinal reinforcement requirement where the reinforcement was terminated at too shallow a depth.	AASHTO LRFD §5.7.3.4 AASHTO LRFD §5.7.4.2, §5.13.4.5.2 AASHTO LRFD §5.8.2.4 TxDOT Bridge Detailing Guide Ch 7 §2 TxDOT Standard Drawing FD Common Foundation Details
09	Main Tower Foundations	§5.2.2 Foundation Cap, 1NT and 1ST	The foundation caps do not meet the project requirements for shear load resistance at the location of the tower legs.	AASHTO LRFD §5.13.3.6, §5.8.1.2, §5.8.1.4, §5.8.3.2, §5.8.3.5, §5.7.3.2, §5.11.2
10	Main Tower	§6.1.4 Main Tower Local Design, 1NT and 1ST	There are several locations in the upper tower at the stay saddle anchorages where the combination of multiple local effects resulted in reinforcement demands exceeded the reinforcement provided.	Design revised, saddles removed ISA checks of revised design in progress.
11	Superstructure	§7.1 Longitudinal Design, Box-Girders	Compressive stress limits are exceeded in the top slab of the northbound box-girder near the pylon.	AASHTO §5.9.4.2.1 AASHTO Table 5.9.4.2.1-1 Compressive stress limits are still exceeded in the top slab of the northbound and southbound box girder near the pylons.
12	Superstructure	§7.1 Longitudinal Design, Box-Girders	Box-girder capacity is exceeded in the strength loading case. Northbound, D/C ~ 1.2; Southbound D/C ~ 1.08. Strength also exceeded for construction case, D/C = 1.2.	AASHTO LRFD §5.7.3 AASHTO LRFD §5.7.4.7 The box girder capacity is exceeded under the Strength loading combination for the segments adjacent to the pylon.
13	Superstructure	§7.1 Longitudinal Design, Box-Girders	Minimum cracking moment (Mcr) is exceeded near midspan in the northbound box-girder.	Issue addressed by the updated design. Additional post-tensioning in the superstructure has been added.
14	Superstructure	§7.3.2.1 Delta Frame, Connection	Deficiency for delta frame CIP connection to bottom block of the box girders	AASHTO LRFD §5.9.4.2.2
15	Superstructure	§7.3.2.2 Delta Frame, Connection	The post-tensioning anchor head passes through a shear friction joint. The anchor head is not designed for this behavior.	AASHTO LRFD §5.10.9.2.3
16	Superstructure	§7.3.2.5 Delta Frame, Diagonal Strut	Maximum allowable stress in concrete in diagonal struts exceeded in service	AASHTO LRFD §5.9.4.2.2
17	Superstructure	§7.3.2.3 Delta Frame, PT Bursting Reinforcement	The vertical bursting reinforcement provided behind the anchors of tendons TD2 and TD3	AASHTO LRFD §5.10.9
18	Superstructure	§7.3.2.4 Delta Frame, Bottom Strut	Bottom strut fails for the Type I delta frame under the Extreme III load combination (loss of stay x2).	AASHTO LRFD §5.7.4
19	Stay Cables	§8 Stay Cables	The loads in the stay cables are exceeding capacity at multiple locations. The envelope of deficient	PTI DC45.1-12 §5.3.3 There are discrepancies between the stay sizes shown in the Erection Manual

ID	Location	Phase 1 Report Section	Phase 1 Project Criteria Deficiency	Applicable Requirements and Notes
			cables is 101 out of 152 where D/C > 1.0	vs. those shown on the drawings. The stay force data tables from the original design have been voided, and they have not been replaced with updated tables based on the updated design. Using stay cable data from either the drawings or the Erection Manual results in some stays having D/C > 1.0.
20	Miscellaneous Items	§9.2 Bearings (Piers 1N/1S and 2N/2S)	The bearings are not able to resist the uplift loading condition observed. The overall system of bearings and vertical stays are incompatible under this loading condition. See Note 4 for additional commentary.	AASHTO LRFD § 14.6.1 The bearings shown in drawing package M13-B+C Rev01 are not able to resist uplift. The capacity of the bearings to resist lateral shear is also compromised under the uplift loading condition.
21	Miscellaneous Items	§9.3 Vertical Stay	The vertical stays do not have adequate capacity to resist the uplift forces once the bearings decompress, and deficient redundancy for loss-of-stay case.	PTI DC45.1-12 §5.3.3 The vertical stays do not provide adequate pre-compression to the bearings to prevent lift-off at Service and Strength loadings. Estimates of stay forces after lift-off give D/C > 1.0 for the vertical stays subject to Strength loadings.

ID	Location	Phase 1 Report Section	Phase 1 Technical Observation	Applicable Requirements and Notes
22	Main Tower Foundations	§5.1 Cable-Stayed Bridge Geotechnical Evaluation, Settlement	The reported settlements in the project geotechnical report are large. The ISA Team performed an independent review and found an appreciable difference (lower values.)	Discretionary The settlement values are large with potential side effects and necessary considerations (e.g. vertical camber of pylon).
23	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT and 1ST axial capacity.	TxDOT foundation criteria does not permit piles to be in tension without prior approval.	Discretionary BDM Ch 4 §2
24	Main Tower Foundations	§5.2.1 Drilled Shafts, 1NT and 1ST, structural detailing	TxDOT foundation criteria has standard detailing that specifies ~~drilled shafts to extend fully into the foundation cap~~ *(sic. intended: "drilled shaft longitudinal reinforcement to extend fully into the bottom of the shaft").* In this case, the shaft reinforcement does not extend far enough into the lower portion of the shaft.	Discretionary TxDOT Geotechnical Manual (2018) Ch 5 §3 TxDOT Standard Drawing FD Common Foundation Details These TxDOT criteria require that the drilled shaft longitudinal reinforcement extend fully to the bottom of the shaft.
25	Back Span Foundations	§5.3.1 Cable-Stayed Bridge Foundation at Piers 1N, 1S, 2N, 2S, Piles	Driven pile strand anchorage details in the foundation cap are vague or not specified.	The original design and the current drawings specify a dead-end anchor for unstressed strand. The drawings state that the details should be approved by the Engineer. It is not clear if

ID	Location	Phase 1 Report Section	Phase 1 Technical Observation	Applicable Requirements and Notes
				"Engineer" refers to TxDOT or the EoR or both. The details remain vague, or they are not specified.
26	Back Span Foundations	§5.3.1 Cable-Stayed Bridge Foundation at Piers 1N, 1S, 2N, 2S, Piles	Driven piles were found to have tension/uplift forces, and TxDOT foundation criteria does not permit this condition without approval.	Discretionary BDM Ch 4 §2
27	Back Span Foundations	§5.3.1 Cable-Stayed Bridge Foundation at Piers 1N, 1S, 2N, 2S, Piles	Calculations for passive strand development into the cap is not specified by AASHTO.	Very little research on development for passive strands exists.
28	Back Span Foundations	§5.3.1 Cable-Stayed Bridge Foundation at Piers 1N, 1S, 2N, 2S, Pile caps	The reinforcement details in the pile cap are unconventional for load transfer where the piles are in significant tension.	The load transfer mechanisms from the pile cap to the piles is indirect, and it is not detailed with anchorage lengths typically used for piles in tension. Various pile cap details do not meet AASHTO LRFD requirements. These details have not been updated in the current design.
29	Main Tower	§6.1.2 Main Tower Flexural Design	Some design checks for the flexural capacity of the pylon give a maximum D/C = 1.04.	This issue may have been resolved with the revised upper tower design. ISA checks of revised design are in progress. However, the ISA team finds that lift 17 has a D/C =1.02 for axial force and bi-axial bending, considering 8 ksi concrete.
30	Main Tower	§6.1.3 Main Tower Shear Design	The design check for the distribution of shear indicated the 3-web portion of the tower section may not adequately distribute the shear to each web within its capacity.	This issue may have been resolved with the revised upper tower design. ISA checks of revised design in progress.
31	Main Tower	§6.2.5 Nodal Zone	The reinforcement detailing within the nodal zone does not have clear load paths for the transfer of load.	Increasing the concrete strength in the nodal zone has improved the situation, and satisfactory load paths have now been identified.
32	Superstructure	§7.1 Longitudinal Design, Vibrations	Vibration checks were performed indicating that the vertical acceleration exceeds the project limit of 0.05g.	TP §13.2.1.12
33	Superstructure	§7.2 Transverse Design, Top Deck	The stresses in the top deck under service loads were found to slightly exceed the code limit.	AASHTO LRFD Table 5.9.4.2.2-1 The stresses in the southbound box girder at the top slab haunch bottom fiber, under Service load combinations still exceed the allowable stress limits.
34	Superstructure	§7.3 Delta Frame	Multiple observations were made related to the delta frame details.	This item involves several discrepancies, missing dimensions, concrete strengths, reinforcement details, grout details, and specifications, etc.

ID	Location	Phase 1 Report Section	Phase 1 Technical Observation	Applicable Requirements and Notes
35	Miscellaneous Items	§9.3 Vertical Stays	The Technical Provisions (13.2.1) prohibit the use of permanent tie-downs. Conditional allowance by TxDOT via an Alternate Technical Concept (ATC) initiated by the Contractor.	Discretionary TP §13.2.1.1
36	Miscellaneous Items	§9.3 Vertical Stays	The drawing details show that the lower anchorages of the vertical stays resemble post-tensioning anchors instead of stay anchors.	The drawings in package MSUB-B Rev00 retain the details of the original design. The tie-downs behave as stays, and their detailing must accommodate the expected longitudinal displacements of the deck relative to the piers.

ID	Location	Phase 2 Part 1 Report Section	New Findings (Since Phase 1)	Applicable Requirements and Notes
37	Superstructure Box Girders	§4.37	Torsional Cracking During Erection	AASHTO LRFD §5.14.2.3.3 The torsion resulting from transversely unbalanced loads presented in the Erection Manual exceeds the box girders' torsional cracking moment, T_{cr}. Also, the principal tensile stresses in the webs are exceeded.
38	Superstructure Delta Frames	§4.38	Loss of 1 Stay of a Pair of Stays	The delta frame vertical bracing element has insufficient flexural capacity to resist the loss of one stay (of a side-by-side pair of stays).
39	Wind Report	§4.39	The ISA team found 4 significant discrepancies in the Wind Report.	Technical Provisions §13.2.1.7
40	General Notes	§4.40	Necessary details regarding segment casting and erection have been removed from the General Notes. New information has been added to the General Notes that requires clarification.	Since the drawings represent formal documentation of the design, it is best to include critical design information on them. It appears that some of the information that existed previously has been removed.
41	Camber	§4.41	Superstructure cambers were not provided in the erection manual.	The ISA team is finding significant differential displacements that will be difficult to control.
42	Back Span Piers Piers 1N, 1S, 2N, 2S Columns	§4.42	The longitudinal and cross-tie reinforcement of the columns do not meet detailing requirements for hollow rectangular compression members.	AASHTO LRFD §5.10.12

Footnotes (text unchanged from Phase 1): Green highlight indicates no longer applicable

Note 1: See Appendix A for ISA assumed procedures for construction staging. Essential data necessary for a comparable erection sequence was not provided by the Design-Build Team.

Note 2: Wind report does not provide enough cases during erection. At least 2 more cases should be included: 1) deck erected up to pier 1N 1S, but before connection at 1N 1S, 2) deck erected up to segments M86 and B86, prior to connection to EJ pier segment at 2N 2S. The number of strands per stay cable assumed in the wind report does not match the ones provided in the final design drawings. No provisions are provided in the wind report for the design of the stay dampers: damping requirements need to be provided for each stay cable.

Note 3: Based on the ISA team's interpretation of the load test results made available to date. We note that 2 of the 3 load tests performed at Tower 1NT failed and that their results were discounted when evaluating the drilled shafts capacity. The ISA team understands the design-build team's view that these two failed tests were not representative of the as-built shafts, because the methodology used for their production differed significantly from the one used to produce the successfully tested shaft and the production shafts. Our evaluation of the foundation capacity is contingent on the veracity of this understanding. In other words, it is assumed that the means and methods used to build the production shafts matched those used for the successful load test, and this must be verified by the Design-Build Team.

Note 4: Significant uplifts at the bearings at Piers 2N and 2S have been observed at the service and the strength limit. These uplifts are not compatible with the connection system composed of the bearing and the vertical tie down (or vertical stay cables). Bearing uplifts are prohibited by the project requirements under the service limit state. At the strength limit state, the current detail would create a significant risk of pounding and vertical relative displacement between the deck and the substructure under the design wind. Increase in the size of the tie down and/or modification of the bearing detail is likely required.

3. Reference Material

The ISA team have performed their work based on information supplied by TxDOT for the CCHB Project. These documents are referenced in the following sub-sections.

It is noted that the relevant version of the codes and manuals are related to the execution date of the project contract, and therefore more recent versions may not be applicable.

Note that the summary of references below are defined on the whole as "project requirements" or "project documents" within this document.

3.1. Specifications and References

The bridge was checked according to the following specifications and standards. *Note that only the documents relevant to the ISA have been listed below.*

3.1.1. Project Specific:

- Texas Department of Transportation "TECHNICAL PROVISIONS FOR US 181 HARBOR BRIDGE PROJECT – Comprehensive Development Agreement", in particular Chapter 8 "Geotechnical", Chapter 13 "Structures" and Attachment 13-1 "Structure Provisions" (*Technical Provisions*)

3.1.2. Texas Department of Transportation:

- TxDOT "Bridge Design Manual – LRFD", revised October 2015 (*TxDOT BDM*)
- TxDOT "Geotechnical Manual", revised December 2012
- TxDOT "Bridge Detailing Guide", August 2014

3.1.3. Design Specifications:

- AASHTO LRFD Bridge Design Specifications, 7th Edition with 2015 interim revision; (*AASHTO LRFD*)
- AASHTO LRFD Bridge Construction Specifications, 3rd Edition 2010
- PTI DC45.1-12: Recommendations for Stay-Cable Design, Testing, and Installation, May 2012 (*PTI DC45.1-12*)
- CEB-FIP "Model Code 1990 – First Edition" *for time dependent concrete material properties only.*

4. Findings

4.1. Redundancy Analysis Report

The project requirements specify that a Redundancy Analysis be performed for the cable stayed bridge. In Phase 1, a cursory report was provided with the project documentation; however it did not fully address the following requirements of the Technical Provisions:

> Developer shall analyze total section fracture at the Extreme Event III limit state for the following Elements:
>
> a) All system-redundant-members;
>
> b) One stay cable;
>
> c) One floor beam; and
>
> d) One edge girder.
>
> Developer shall prepare a Redundancy Report that validates the design of the New Harbor Bridge satisfies the requirements for redundancy. At a minimum, the report shall include the following:
>
> a) Procedures and methodologies used to achieve redundancy of the New Harbor Bridge;
>
> b) Procedures used to calculate the magnitude of the fracture dynamic force and how it was applied;
>
> c) Calculation results validating the achievement of redundancy;
>
> d) Graphic representations depicting structural details described in the report; and
>
> e) Summary identifying all members of the New Harbor Bridge in tension and how they achieve redundancy.
>
> The Redundancy Report shall be signed and certified by the Lead New Harbor Bridge Design Engineer. Developer shall submit the Redundancy Report to TxDOT. The Redundancy Report shall be subject to TxDOT Approval.

4.1.1. General

An updated Redundancy Report has been submitted by the Design-Build team (Document No. 277609-NHB-REP-MRDND-01 dated April 6, 2021); however, this report is incomplete. In Section 5.1.1, the updated report says, "At the time of issue of this Redundancy Report, the design verification of the upper tower is not complete."

Also, the updated report mentions in the sections below that only a limited number of locations have been investigated:

- Section 3.2.1 – Loss of a single stay cable – only 8 locations considered
- Section 3.2.2 – Anchor box web fracture – only 3 locations considered
- Sections 3.2.3 and 5.3.1 – Anchor box transverse beam fracture – only 4 locations considered
- Sections 3.2.4 and 5.4.1 – Vertical tendon strand loss – only 4 locations (3 stay sizes) considered

In Section 2.3, it is stated that the loss of a delta frame has been considered as an additional redundancy objective investigated. However, the loss of a delta frame is considered at only 4 locations. Various elements subjected to redistribution have now been considered; however, these checks have not been comprehensive. For example, the report does not address the effect of the loss of one delta frame on adjacent delta frames. Also, the updated Redundancy Report does not address the potential loss of internal box girder struts or the external struts connecting the box girders to the towers.

The updated Redundancy Report compares demands considered from Extreme events to Strength demand envelopes. Presumably, the Strength Envelopes have been checked vs. capacities, which may be documented in other reports or calculations. This approach may be acceptable if Strength envelopes have been checked, and Extreme effects are less than Strength effects. However, the updated report does not adequately address locations where Extreme effects exceed Strength effects. For example, Appendix A2.1 (p. A7) shows that Extreme demands for shear (Vz), moment (My), and torsion (MT) exceed Strength demand envelopes at the top of the tower. The report does not present calculations of capacity, nor does it include comparisons of concomitant demands (axial load, shear, moment, and/or torsion interaction) with structural element capacities. In other words, there are no demand-to-capacity ratios, no demand ≤ capacity checks, and no interaction diagrams showing demands plotted within capacity curves or surfaces for this case. The text on Page A7 suggests that perhaps more will (or was intended to) follow:

> "Where demands fall outside of the strength envelopes at the upper tower, verification will be performed in the Released for Construction documents and submitted with the respective calculation package. Note that these results are conservatively based on a dynamic amplification factor of 2.0, but could be reduced to 1.5 in accordance with PTI recommendations, as allowed by TP 13.2.1.3."

The reduction of the dynamic amplification factor must be justified by analyses. However, the ISA team has not found such analyses (see §4.1.3 below for more on the dynamic amplification topic).

In some sections of the updated Redundancy Report appendices, it is stated that "Where demands fall outside the envelopes, spot checks are performed to verify that demand-to-capacity ratios are less than 1.0." The expression "spot check" implies that such checking was not comprehensive. However, this cannot be assessed, since the spot check calculations were not presented. In some cases, a utilization percentage is presented; however, in such instances the governing load event, location, demand, and capacity have been omitted.

No time-history analyses have been presented, even though the Technical Provisions §13.2.1.3 requires, "The FDF for other Elements shall be developed based on the results of a 3-D dynamic (time history) model."

Considering the paragraphs above, the Redundancy Report is not complete and it is not sufficiently comprehensive with regard to checking "a) <u>All</u> system-redundant-members" and "c) Calculation results validating the achievement of redundancy" as required by the Technical Provisions §13.2.1.3.

In addition, the ISA team disagrees with numerous opinions presented in the updated Redundancy Report. However, for brevity, ISA disagreements with such opinions are not exhaustively presented in this Part 1 report, with the notable and important exceptions described in the next two sections below.

4.1.2. Tie-Downs

The updated report does not consider the loss of an entire vertical tie-down stay. Rather, it suggests that only the loss of a single strand was considered. This presumes that the tie-down behaves as post-tensioning; however, this is not the case. The tie-downs are mainly tensile elements; however, they are also subjected to bending stresses resulting from the longitudinal displacement of the deck relative to the piers. Also, fracture of an anchor head or trumplate, which has actually occurred in other stay cable structures, would result in the loss of an entire set of strands – not just one. The tie-downs must be considered as stays because they behave like stays, and the loss of an entire tie-down stay must be considered in the Redundancy Report, as tie-down failure could result in progressive collapse.

4.1.3. Tower Anchor Boxes

Section 4.1.2 of the updated Redundancy Report states that a dynamic amplification factor (DAF) of 2.0 was applied for the stay loss events considered, except for the tower anchor box webs where the report states that no dynamic effects were considered. The updated Redundancy Report suggests that the DAF (or FDF for Fracture Dynamic Force as defined in the Technical Provisions) may be reduced from 2.0 to 1.5. However, reduction to 1.5 is only permitted by PTI DC45.1-12 §5.5 if justified by non-linear dynamic analyses (time-history analyses), which have not been presented in the updated Redundancy Report.

This is important because Phase 1 ISA calculations revealed demand-to-capacity ratios in excess of 1.0 when considering loss of stay load combinations. The updated design revised the stay connection from saddles to anchorages at the upper towers. Additional anchorages in the towers improve redundancy; however, the anchor boxes themselves are critical. The ISA team does not agree with neglecting dynamic amplification for anchor box members without validation.

The overall dimensions of the upper tower did not increase in the updated design, even though anchor boxes have replaced saddles.

For the reasons summarized above, the ISA still finds that the design does not meet the requirements of Technical Provisions §13.2.1.3. Tower anchor box failure could result in progressive collapse, and so the failure of a tower anchor box subject to the proper loading must be considered in the Redundancy Report. The ISA is still evaluating the updated design, and will present any additional findings related to the Redundancy Report in the ISA Phase 2 Part 2 report.

4.2. Construction Sequence

AASHTO LRFD §2.5.3 requires that a predefined construction sequence be included as part of the final design documents. For the original design, this documentation was missing, so the ISA needed to make assumptions about the construction sequence for its Phase 1 work.

The Design-Build team has now submitted an Erection Manual that addressed Finding 02 and the AASHTO LRFD §2.5.3 requirement, so this Phase 1 finding may be considered resolved. However, the ISA team has identified other findings related to the current construction sequence documents. See Finding 38 and Finding 41.

4.3. Wind Report

Per the original ISA evaluation, the Wind Report did not consider enough cases during erection, which violates Technical Provisions § 13.2.1.7.1 and §13.2.1.7.3 where wind effects shall be evaluated "during critical construction stages". To capture the critical cases, the original report should have considered at least 2 more cases: 1) deck erected up to pier 1N 1S, but before connection at 1N 1S, 2) deck erected up to segments M86 and B86, prior to connection to EJ pier segment at 2N 2S.

The Wind Report has been updated (the current version is MWER Rev01), and Sections 4 and 5 of the revised report adequately address the original ISA findings noted above. Therefore, this specific issue may be considered resolved.

New issues have been identified concerning the information in the updated Wind Report and the updated design. See Finding 39.

4.4. Supplementary Driven Piles at 1NT

In the foundation supporting the tower at 1NT, supplementary piles were driven and installed adjacent to the drilled shafts. Based on a detailed non-linear analysis using FLAC3D software, it was demonstrated during Phase 1 that the driven piles provide very little additional support to the foundation. Also, because of the pile detailing, the reliability of any additional capacity was questionable. Upon confirming that the piles did not inadvertently reduce the geotechnical capacity of the drilled shaft foundation, these supplementary driven piles have been disregarded for ISA design checks.

According to the current design documents, these supplemental piles are no longer considered by the current Design-Build team, so this Phase 1 finding may be considered resolved.

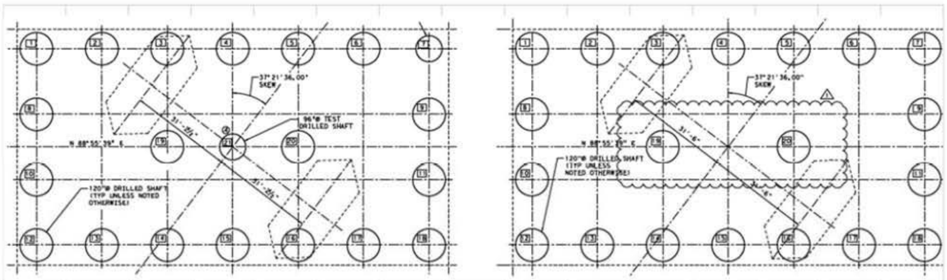
4.5. Tower Drilled Shafts – Insufficient Capacity

Multiple drilled shafts do not meet the project requirements for geotechnical axial load resistance. The following project requirements apply:

- Technical Provisions § 13.2.1.14
- TxDOT BDM Ch 2 §1
- TxDOT Geotechnical Manual Ch 5 §2, Ch 5 §3
- AASHTO LRFD §10.5.5.2.4

At foundation 1NT, 16 of the 20 drilled shafts are deficient for Service and for Strength and Extreme load combinations. At foundation 1ST, 12 of 20 drilled shafts are deficient under Service loads and 10 of 20 are deficient under Strength and Extreme load combinations. Figures 2 and 3 below show the maximum compression observed in each drilled shaft along with D/C ratios for each shaft for Service, Strength, and Extreme load combinations.

SERVICE LOAD COMBINATIONS - RESULTS



1NT **1ST**

CASE 0: MAX COMPRESSION IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS), KIPS

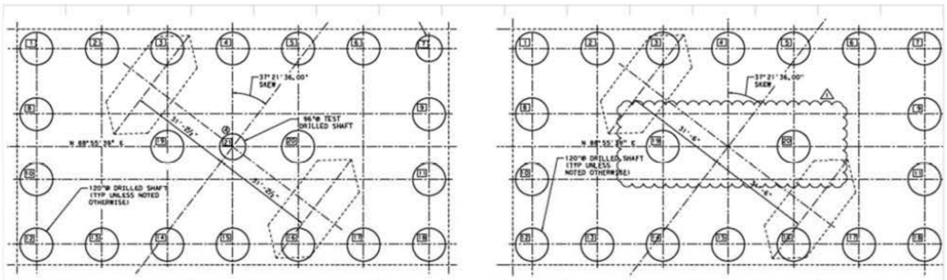
-11,303	-12,522	-13,118	-12,889	-11,786	-10,188	-5,803		-11,433	-13,337	-14,372	-14,123	-12,567	-10,484	-7,981
-9,767						-8,569		-9,446						-8,756
		-10,655		-10,684						-10,870		-11,096		
-8,719						-9,852		-7,844						-9,952
-8,351	-10,408	-12,023	-13,198	-13,546	-12,904	-11,749		-6,440	-9,161	-11,473	-13,250	-13,869	-12,961	-11,205
1NT								**1ST**						

CASE 0: D/C RATIO

1.20	1.33	1.39	1.36	1.25	1.08	0.81		1.06	1.23	1.33	1.31	1.16	0.97	0.74
1.03						0.91		0.87						0.81
		1.13		1.13						1.01		1.03		
0.92						1.04		0.73						0.92
0.88	1.10	1.27	1.40	1.43	1.37	1.24		0.60	0.85	1.06	1.23	1.28	1.20	1.04
1NT								**1ST**						

Figure 2: Summary of Service Axial Compression and D/C Ratios for 1NT/1ST Drilled Shafts – Drilled Shaft Layout Schematic Provided by Others

STRENGTH AND EXTREME LOAD COMBINATIONS - RESULTS



1NT 1ST

CASE 0: MAX COMPRESSION IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS), KIPS

-15,864	-16,294	-16,450	-16,192	-15,376	-14,263	-9,551		-16,469	-17,254	-17,569	-17,419	-16,408	-14,857	-12,307
-14,254						-12,269		-14,164						-12,216
		-13,690		-13,904						-13,874		-14,034		
-12,069						-14,503		-11,195						-14,315
-12,721	-14,123	-15,414	-16,435	-16,808	-16,650	-16,217		-10,043	-12,973	-15,230	-16,654	-17,230	-16,938	-16,254
1NT								1ST						

CASE 0: D/C RATIO

1.20	1.23	1.25	1.23	1.16	1.08	0.95		1.09	1.14	1.16	1.15	1.09	0.98	0.82
1.08						0.93		0.94						0.81
		1.04		1.05						0.92		0.93		
0.91						1.10		0.74						0.95
0.96	1.07	1.17	1.25	1.27	1.26	1.23		0.67	0.86	1.01	1.10	1.14	1.12	1.08
1NT								1ST						

Figure 3: Summary of Strength and Extreme Axial Compression and D/C Ratios for 1NT/1ST Drilled Shafts – Drilled Shaft Layout Schematic Provided by Others

Based on the ISA team's interpretation of the load test results made available to date, we note that 2 of the 3 load tests performed at Tower 1NT failed and that their results were discounted when evaluating drilled shaft capacity. The ISA team understands the design-build team's view that these two failed tests were not representative of the as-built shafts, because the methodology used for their production differed significantly from the one used to produce the successfully tested shaft and the production shafts. The ISA team's evaluation of the foundation capacity is contingent on the veracity of this understanding. In other words, it is assumed that the means and methods used to build the production shafts matched those used for the successful load test, and this must be verified by the Design-Build Team.

Also, drawing package "277609-NHB-PLN-M02-02" includes updated drawings for drilled shaft types A-F; however, it does not include a drawing for drilled shaft type G.

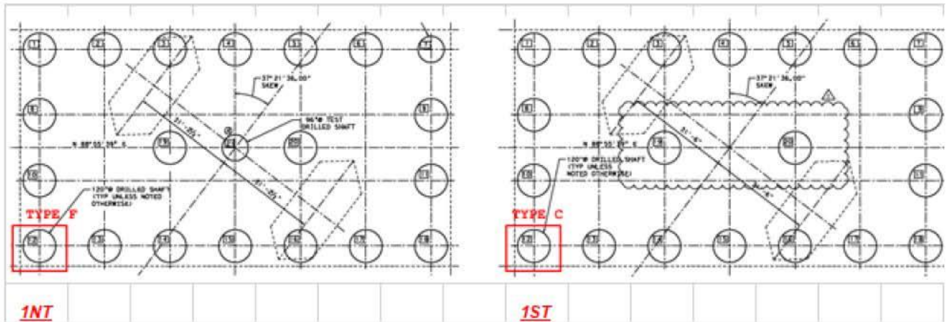
4.6. Tower Drilled Shafts – Invalid "Overstress" Allowance Applied

The use of a 1.33 "overstress" allowance factor is not supported by the project technical requirements.

According to TxDOT, this issue had been resolved, and presumably it was agreed that the overstress allowance considered by the original Engineer-of-Record (EoR) did not apply. The main body text of the updated Geotechnical Report no longer references a 33% overstress allowance. Specifically, it was removed from the text in §5.4. However, in Appendix 4.1.3 of the updated Geotechnical Report, a reference to a 33% increase remains, and only the IBC code citation that was included in the original Geotechnical Report has been removed from the text in the updated Geotechnical Report. Also, the drilled shaft drawings in package M02 Rev02 still mention a 33% stress allowance in the notes and a 1.33 factor is still included in the capacities reported on these drawings.

4.7. Tower Drilled Shafts – Insufficient Structural Capacity

One drilled shaft at Tower 1NT and one drilled shaft at Tower 1ST do not meet the project requirements for combined axial load / flexural resistance per AASHTO LRFD §5.7.4.5. This occurs under combinations that result in tension in corner drilled shafts. Figure 4 below provides a summary of locations and D/C ratios associated with this finding.



CASE 0: MAX FLEXURE D/C RATIO IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS)														
0.90	0.73	0.72	0.71	0.74	0.76	0.78		0.83	0.77	0.79	0.79	0.82	0.86	0.92
0.82						0.80		0.78						0.84
		0.69		0.65						0.64		0.69		
0.90						0.79		0.75						0.82
1.45	0.93	0.87	0.76	0.76	0.75	0.89		1.43	0.83	0.72	0.73	0.76	0.74	0.80
1NT								1ST						

Figure 4: Summary of Maximum Axial Load - Flexural D/C Ratios for the 1NT and 1ST Drilled Shafts – Drilled Shaft Layout Schematic Provided by Others

The capacity at critical sections are reduced due to shallow termination of longitudinal reinforcement in the shaft. Noting that the deficient drilled shafts are at tension locations, Figure 5 below shows that the demand-to-capacity is critical at a lower position in the shaft (~-64 ft) where the reinforcement is discontinued at a depth that is too shallow, given the demands in these two drilled shafts at those depths.

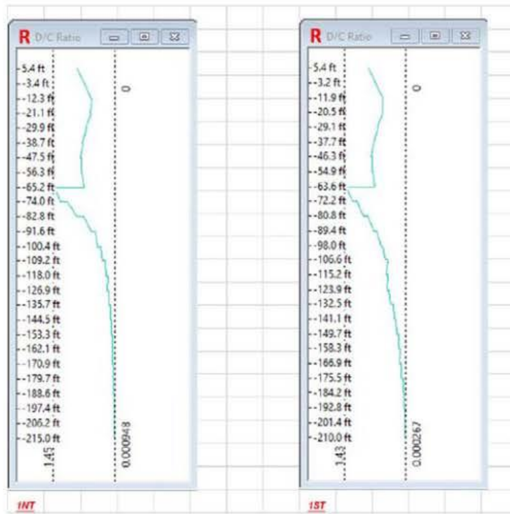


Figure 5: Plot of Drilled Shaft Axial Load - Flexural D/C Ratios Over Depth of Drilled Shafts

4.8. Tower Drilled Shafts – Deficient Reinforcement Detailing

Multiple drilled shaft locations have deficient detailing that is not meeting the project requirements. AASHTO LRFD includes several detailing requirements for the drilled shafts; in particular: minimum transverse reinforcement, control of cracking by distribution of reinforcement, lap splice lengths, longitudinal reinforcement ratio, and extents of longitudinal reinforcement. Several of these requirements are not satisfied, as described in the following sections:

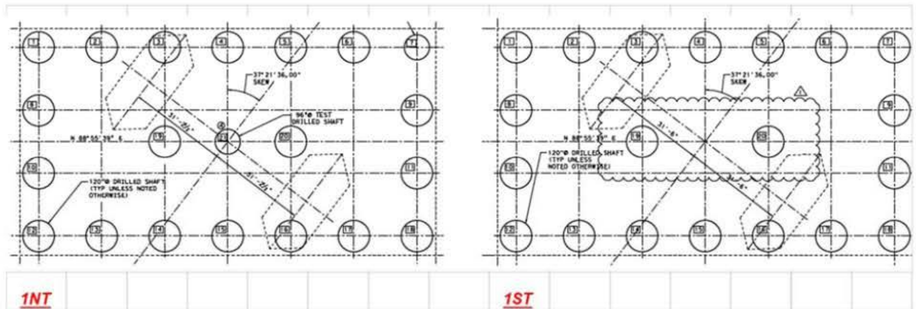
4.8.1. Drilled Shaft Detailing -- Minimum Transverse Reinforcement

AASHTO §5.8.2.4 requires transverse reinforcement to be provided when the shear demand in the Strength limit state exceeds one-half of the factored concrete shear capacity.

The factored concrete shear capacity is calculated as per AASHTO §5.8.3.4.2 using the General Procedure (i.e. "modified compression field theory"), and is a function of the loading (shear, moment, and axial force) as well as the reinforcement provided (both longitudinal and transverse).

Verification of minimum transverse reinforcement is performed by calculating a C/D (capacity / demand) ratio, which is equal to the provided transverse reinforcement divided by the required transverse reinforcement.

Verification was conducted along the entire depth of the drilled shafts (i.e. from top to bottom). The minimum transverse reinforcement C / D ratio in each of the drilled shafts is presented in Figure 6 below for both 1NT and 1ST:



CASE 0: MIN C / D RATIO IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS)														
1.22	999	999	999	999	0.31	999		999	999	999	999	999	999	1.22
999						999		999						999
		999		999						999		999		
999						999		999						999
0.31	999	999	999	999	999	999		0.31	0.31	999	999	999	999	1.22
1NT								1ST						

Figure 6: Summary of Minimum Transverse Reinforcement C/D Ratios for Drilled Shafts – Drilled Shaft Layout Schematic Provided by Others

Note that a C / D ratio of 999 presented in Figure 6 above indicates that the shear demand is less than one-half the factored concrete shear capacity, and therefore transverse reinforcement is not required as per AASHTO §5.8.2.4 at this location.

For two drilled shafts at 1NT and two drilled shafts at 1ST, the transverse reinforcement provided is less than the minimum required transverse reinforcement required.

4.8.2. Drilled Shaft Detailing -- Control of Cracking by Distribution of Reinforcement

Verification of the control of cracking by distribution of reinforcement in the drilled shafts was performed in accordance with AASHTO LRFD §5.7.3.4.

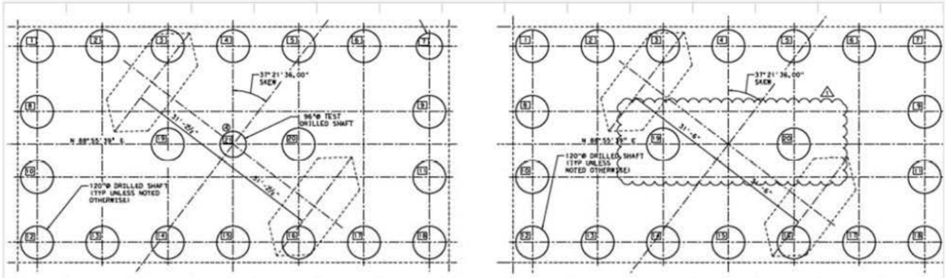
The reinforcement stress demand is obtained using the computer program X8008 (written by the U.S. Army Engineer Waterways Experiment Station), which performs a sectional stress analysis of combined axial load plus bi-axial flexure.

The allowable reinforcement stress *fss* is calculated in accordance with AASHTO equation 5.7.3.4-1. In this equation, the exposure factor γe is taken to be 0.75, based on the PSI Geotechnical Engineering Services report, which indicates that the foundation soils are corrosive.

Using the reinforcement stress demand and allowable reinforcement stress, an "equivalent" D/C ratio for control of cracking by distribution of reinforcement is calculated as follows:

D/C ratio = reinforcement stress demand / allowable reinforcement stress

The maximum reinforcement stress demand in each of the tower drilled shafts and the corresponding equivalent D/C ratios are presented in Figure 7 below for both 1NT and 1ST:



1NT **1ST**

CASE 0: MAX REINFORCEMENT STRESS IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS), KSI

16.39	1.44	0.27	0	0	3.03	40.12		1.88	0	0	0	0.85	4.81	25.44
5.43						4.29		1.83						9.03
		0		0						0		0		
8.28						2.63		7.06						5.67
37.01	4.6	0.82	0	0	0	3.27		61.64	9.15	0.7	0	0.73	1.92	10.75
1NT								**1ST**						

CASE 0: D/C RATIOS

0.53	0.05	0.01	0.00	0.00	0.10	1.11		0.06	0.00	0.00	0.00	0.03	0.15	0.82
0.17						0.14		0.06						0.29
		0.00		0.00						0.00		0.00		
0.27						0.08		0.23						0.18
1.19	0.15	0.03	0.00	0.00	0.00	0.11		1.99	0.29	0.02	0.00	0.02	0.06	0.35
1NT								**1ST**						

Figure 7: Maximum Longitudinal Reinforcement Stress and Equivalent D/C Ratios for Control of Cracking by Distribution of Reinforcement – Drilled Shaft Layout Schematic Provided by Others

4.8.3. Longitudinal Reinforcement Ratio

Verification of the drilled shaft longitudinal reinforcement ratio was performed in accordance with the AASHTO LRFD Bridge Design Specifications, the TxDOT Bridge Detailing Guide, and the TxDOT Common Foundation Details (FD) drawing.

The provided longitudinal reinforcement ratio along the depth of the drilled shaft (from top to bottom) is summarized in Figure 8 below for drilled shaft types A-G:

A, ϕ = 10-ft		B, ϕ = 10-ft		C, ϕ = 10-ft		D, ϕ = 10-ft		E, ϕ = 10-ft		F, ϕ = 10-ft		G, ϕ = 8-ft	
Length FT	Reinf. Ratio	Length FT	Reinf. Ratio	Length FT	Reinf. Ratio	Length FT	Reinf. Ratio	Length FT	Reinf. Ratio	Length FT	Reinf. Ratio	Length FT	Reinf. Ratio
90	0.83%	15	0.83%	62	0.83%	90	0.83%	15	0.83%	62	0.83%	15	1.29%
115.0313	0.41%	37	1.24%	143.0313	0.41%	120.0313	0.41%	37	1.24%	148.0313	0.41%	47	1.94%
		15	0.83%					15	0.83%			15	1.29%
		138.0313	0.41%					143.0313	0.41%			133.0313	0.65%
205.0313		205.0313		205.0313		210.0313		210.0313		210.0313		210.0313	

Figure 8: Drilled Shaft Reinforcement Ratio

The minimum provided longitudinal reinforcement ratio is 0.41%, used in the bottom of drilled shaft types A-F. As mentioned in §4.5 above, the updated design does not include a drawing for drilled shaft type G. Assuming that it still has the same design as the original design, the drilled shaft Type G has a longitudinal reinforcement ratio of 0.65% in the bottom portion.

The minimum required longitudinal reinforcement ratio is calculated using AASHTO equation 5.7.4.2-3. For concrete with a compressive strength of 3.6 ksi, this equation yields a minimum ratio of 0.81%. Additionally, AASHTO §5.13.4.5.2 requires a minimum of 0.8% for drilled shafts.

The TxDOT Bridge Detailing Guide (Table 7-1 excerpted below) and the TxDOT FD drawing show a typical longitudinal reinforcement ratio of ~1% in drilled shafts:

Table 7-1: Drilled Shaft Reinforcing

Shaft Diameter	Vertical Reinforcing		
	No. of Bars	Size	%
18"	6	#6	1.04%
24"	8	#7	1.06%
30"	8	#9	1.13%
36"	10	#9	0.98%
42"	14	#9	1.01%
48"	18	#9	0.99%
54"	16	#11	1.09%
60"	20	#11	1.10%
66"	22	#11	1.00%
72"	26	#11	1.00%

The provided longitudinal reinforcement ratio in the lower portion of the drilled shafts (0.41% for Types A-F and 0.65% for Type G) is less than the AASHTO requirements for minimum longitudinal reinforcement ratio (0.81%), as well as TxDOT detailing criteria (~1%).

4.9. Tower Foundation Cap – Insufficient Capacity

The tower foundation caps do not meet the project requirements for flexure and shear load resistance at the location of the tower legs. The sections below describe the potential for a **brittle failure of the foundation cap during an Extreme wind event that would lead to a sudden collapse of the bridge with little or no warning.**

4.9.1. General

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 tower is skewed to the foundation by a ~37.4° angle. This causes irregular loading and requires more detailed modeling than a traditional cap. A plan view of the foundation cap layout is shown in Figure 9 below for both 1NT and 1ST.

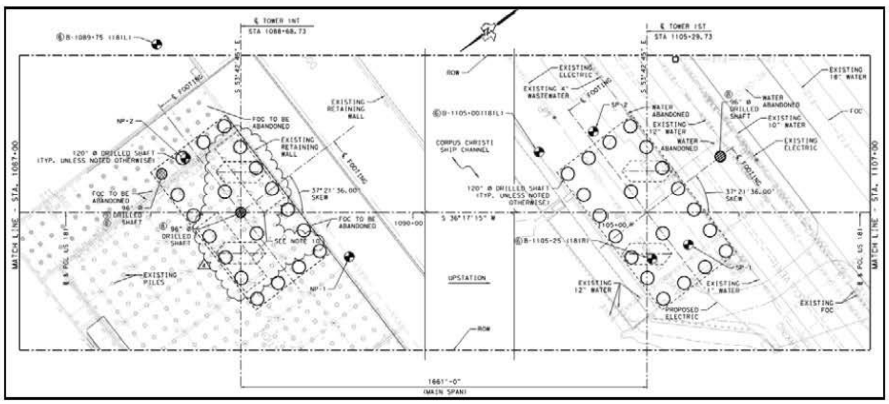


Figure 9: Plan View of Main Tower Foundation Caps at 1NT and 1ST – Schematic Provided by Others

The foundation cap is 132'-0" wide, 72'-0" long and 18'-0" deep with a conventional reinforcement pattern orthogonal to the primary direction of the cap, and a regular spacing of shear stirrups. Supplemental reinforcement is included oriented with respect to the tower legs.

Sample plots of the reinforcement for 1ST (1NT similar) are presented in Figures 10 and 11 below.

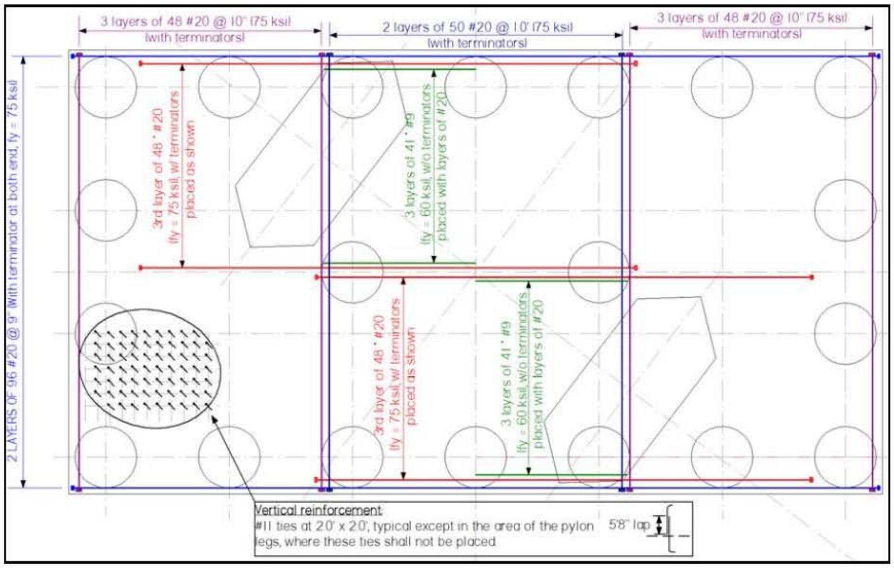


Figure 10: Foundation Cap 1ST, Bottom Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

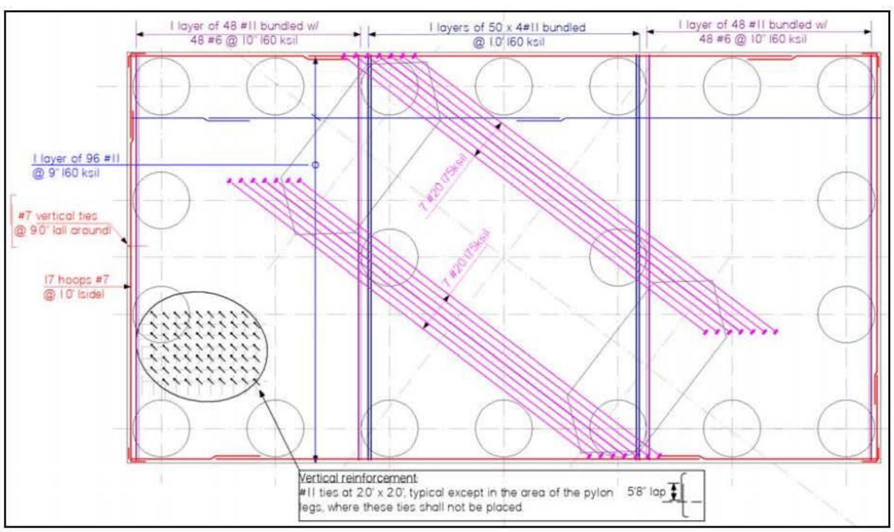


Figure 11: Foundation Cap 1ST, Top Mat Reinforcement – Drilled Shaft Layout Schematic Provided by Others

Demands in the cap were determined using a thick plate model, as shown in the plot in Figure 12 below.

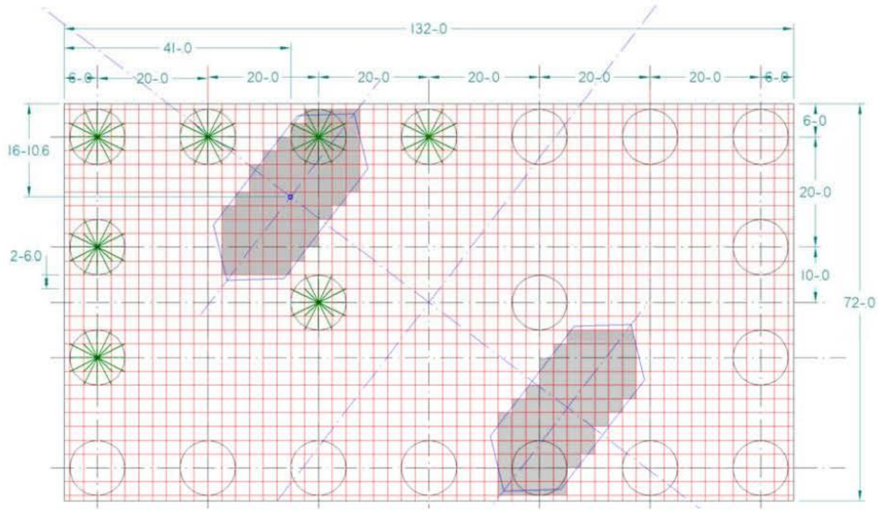


Figure 12: Foundation Cap 1ST, Plate Model

Equivalent loads were developed that were applied to the tower legs and the drilled shafts to model the axial load, shear, and bending inputs. The loading was generated by the program, FB-MultiPier, and equilibrium in the cap was verified. Rigid elements were used in the vicinity of the tower legs and drilled shafts to capture the plate stiffening effect of these elements. This is shown graphically in Figure 13 below. The elements used in this analysis allowed for thick plates, considered shear deformations, and provided out-of-plane shear results.

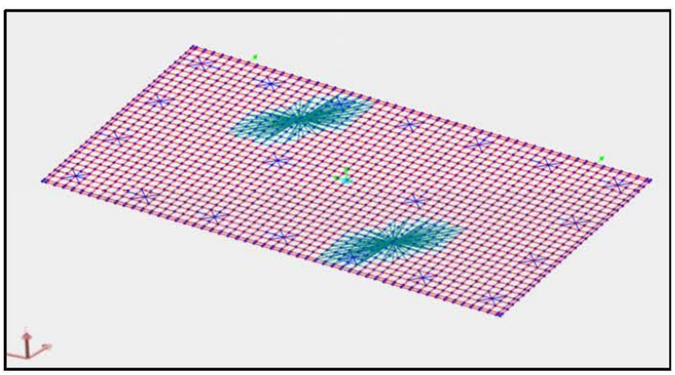


Figure 13 Foundation Cap 1ST, Plate Model

4.9.2. Structural Design Checks

The foundation cap was checked for multiple design requirements. For brevity, only the following are presented herein.

- Transverse bending moment in the foundation cap
- One-way shear in the foundation cap
- Two-way shear in the foundation cap
- Stut-and-tie modeling of the foundation cap

4.9.2.1. Transverse Bending Moment in Foundation Cap

The flexural sections checked for transverse bending moment are shown in Figure 14 below.

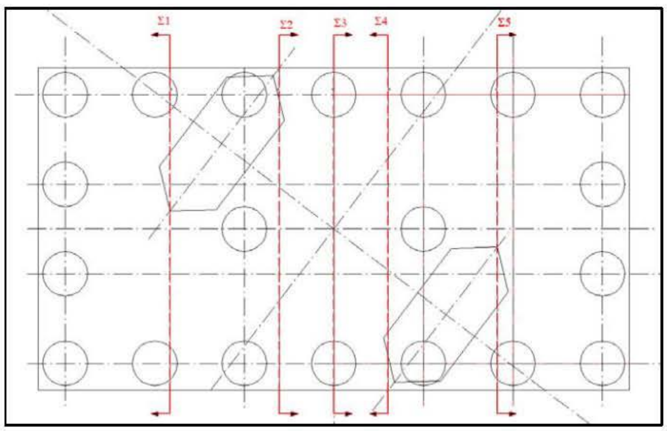


Figure 14: Foundation Cap 1ST, Section Cuts Considered for Transverse Bending Moments

The sections Σ_1 to Σ_5, the ½ sections Σ_{1N} to Σ_{5N} and the ½ sections Σ_{1S} to Σ_{5S} have been checked for Strength and Extreme combinations based on AASHTO LRFD requirements.

Critical sections chosen were verified by a visual inspection of the demand in the cap. Sections Σ_1, Σ_2, Σ_4 and Σ_5 are located at the critical faces of the tower legs. This is represented by the plot in Figure 15 below showing transverse bending moment throughout the 1ST foundation cap. The foundation cap at 1NT is similar.

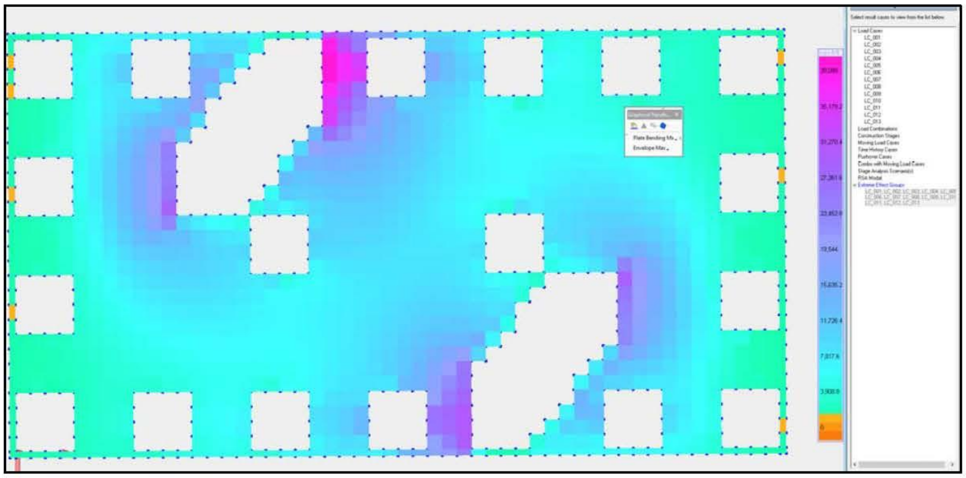
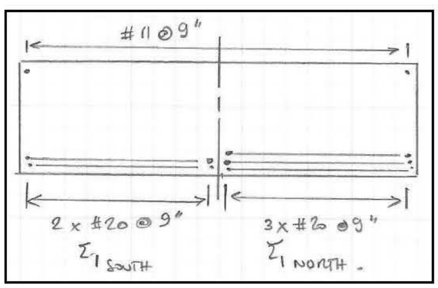


Figure 15: Foundation Cap 1ST, Plate Demands for Transverse Bending Moment

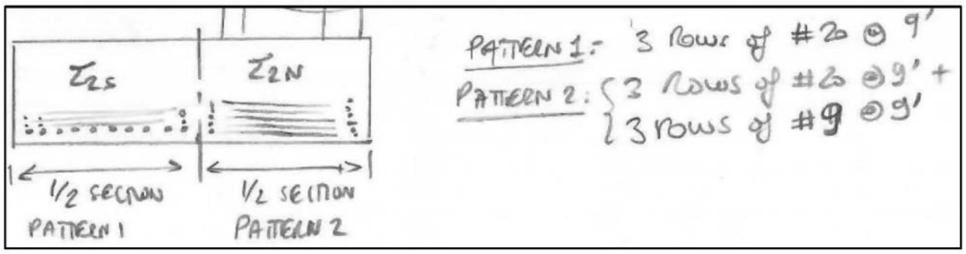
A summary of results is shown in the following plots (Figures 16 – 20). The D/C ratio was checked considering both the total section width and the half-section width, which better captures the localized demands generated by the tower leg and matches the bending reinforcement pattern provided in the design.



Σ1 Axial + Bending – Strength – Result Summary		
Section	Governing LC	Max D/C ratio
Σ1 North	10	0.70 → OK
Σ1 South	10	0.32 → OK
Σ1 total	10	0.55 → OK

Figure 16: Foundation Cap 1NT and 1ST, Summary of Transverse Checks at Section 1

The Section 2 check is shown below, which is similar to Section 4 but with greater demand. Therefore, the check at Section 2 suffices as a check for both sections.



Σ2 Axial + Bending – Strength Limit state – Result Summary		
Section	Governing LC	Max D/C ratio
Σ2 North	2	1.05 → DEFICIENT
Σ2 South	10	0.33 → OK
Σ2 total	2	0.67 → OK

Figure 17: Foundation Cap 1NT and 1ST, Summary of Transverse Checks at Section 2

The reinforcement development length of the pile cap at Section 2 was calculated and found to be deficient per AASHTO LRFD §5.11.1.2.

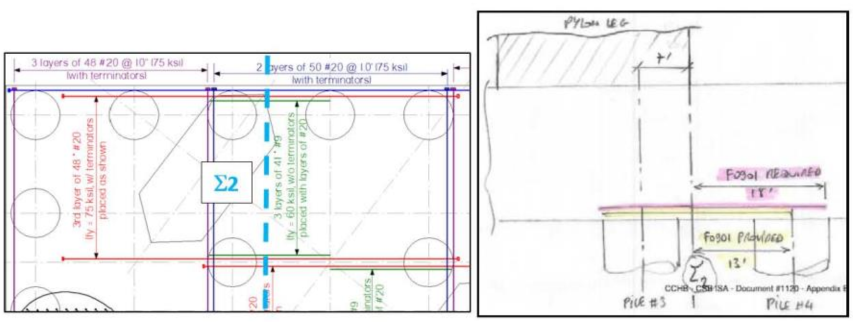
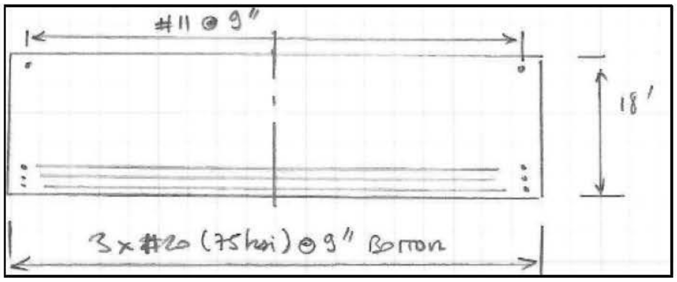


Figure 18: Foundation Cap 1NT and 1ST, Insufficient Reinforcement Development at Section 2



Σ3 Axial + Bending – Strength Limit state – Result Summary		
Section	Governing LC	Max D/C ratio
Σ3 North	2	0.69→ OK
Σ3 South	12	0.36 → OK
Σ3 total	2	0.5→ OK

Figure 19: Foundation Cap 1NT and 1ST, Summary of Transverse Bending Moment Checks at Section 3

Σ5 Axial + Bending – Strength Limit state – Result Summary		
Section	Governing LC	Max D/C ratio
Σ5 total	9	0.52→ OK

Figure 20: Foundation Cap 1NT and 1ST, Summary of Transverse Bending Moment Checks at Section 5

Similar calculations were performed for plate bending moments in the longitudinal direction, and the sections checked were found to be adequately reinforced. Plots and summary results for these checks may be found in the ISA Phase 1 report.

Most of the sections checked for transverse bending moment generally satisfy AASHTO LRFD §5.7.3.2. However, the half-section at Section 2 has a D/C ratio = 1.05, and the reinforcement to resist this transverse bending moment at this location is not sufficiently developed. Concern at this location is exacerbated by the one-way and two-way shear problems discussed in Sections §4.9.2.2, which follows below.

4.9.2.2. One-Way Action and Two-Way Action in Foundation Cap

The shear resistance of the foundation cap for Tower 1ST has been determined following the requirements of AASHTO LRFD § 5.13.3.6 "Shear in Slab and Footing":

"In determining the shear resistance of slab and footing in the vicinity of concentrated loads or reaction forces, the more critical of the following conditions shall govern:

- *One-way action, with a critical section extending in a plane across the entire width and located at a distance taken as specified in article §5.8.3.2.*
- *Two-way action, with a critical section perpendicular to the plane of the slab and located so that its perimeter bo is a minimum but no closer than 0.5dv to the perimeter of the concentrated load or reaction area."*

A graphical representation of the critical sections for shear in footings is shown in Figure 21 below, noting that two-way action is taken at 0.5dv (rather than dv) per the AASHTO LRFD requirement stated above.

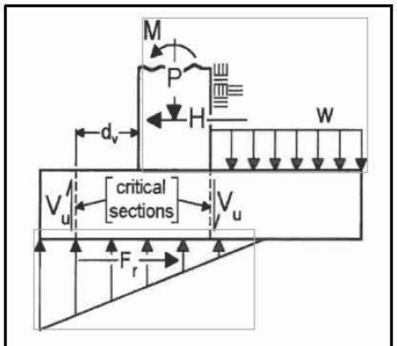


Figure C5.13.3.6.1-1—Example of Critical Section for Shear in Footings

Figure 21: General Plot of Critical Shear Sections for Evaluation in Foundation Cap

In accordance with the AASHTO LRFD requirements stated above, the sections to check under one-way action are Σ_1', Σ_3 and Σ_5', as shown in Figure 22 below. Note: Σ_2 is not considered for one-way action due to its proximity to the edge of the tower leg (within dv/2).

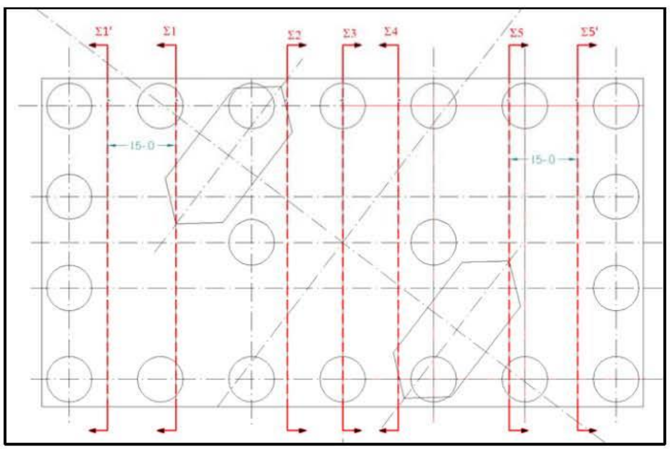


Figure 22: Plot of Critical Shear Sections for Evaluation in Foundation Cap

As observed with the flexural demands in the foundation cap, the shear loads are highly concentrated adjacent to the tower legs. For conventional designs, AASHTO may allow for the full-section width to be considered as part of the capacity, but that generally applies to a concentrically loaded cap without pronounced edge loadings, which is not the case for the foundation caps at Towers 1NT and 1ST. Therefore, both full- and half-sections are checked for one-way shear.

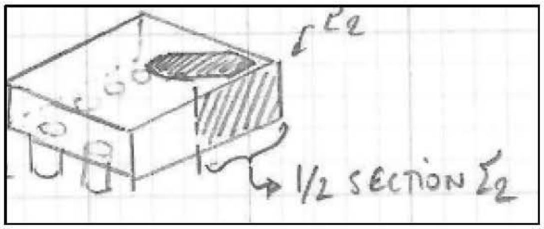


Figure 23: Sketch Showing Concentrated Reaction from Tower Leg at the Edge of the Foundation Cap

A summary of results for one-way action is provided below. If the entire width of the cap could be mobilized to resist the demand, it would have sufficient capacity. However, considering the half-width, which is more appropriate considering actual behavior, the cap does not have sufficient capacity.

One-way shear demand capacity ratio – transverse shear - full-section			
Section	**Vu (kips)**	**Vr (kips)**	**D/C ratio**
Σ_1' - full	38,573.0	43,796	0.88→OK
Σ_3- full	23,542.0	43,796	0.54→OK
Σ_5' - full	35,865.0	43,796	0.82→OK

One-way shear demand capacity ratio – transverse shear – half-section			
Section	**Vu (kips)**	**Vr (kips)**	**D/C ratio**
Σ_1' – half-section	24,434.0	21,900	1.11→DEFICIENT
Σ_3- half-section	28,137.0	21,900	1.28→DEFICIENT
Σ_5' - half-section	20,889.0	21,900	0.95→OK

Longitudinal shear is taken half-way between the pylon legs.

One-way shear demand capacity ratio – longitudinal shear – half-section			
Section	**Vu (kips)**	**Vr (kips)**	**D/C ratio**
Σ_B & Σ_C - half-section	36,968	40,086	0.92→OK

For two-way action, multiple locations were evaluated to verify the structural capacity. Cases considered at locations in the foundation cap around the drilled shafts were found to be adequately reinforced. However, for locations around the tower leg, there is insufficient capacity to resist two-way action. In this case, axial load and bending moment from the tower leg apply localized stresses normal to the cap concentrated at the "pointy toe" of the tower leg near the edge of the cap, as shown in Figure 24 below.

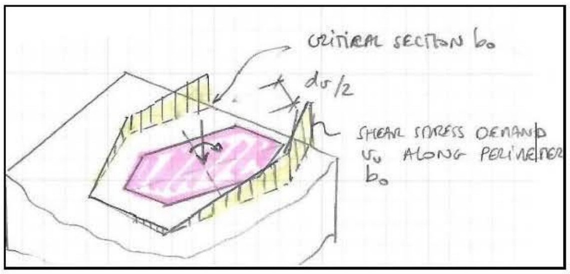


Figure 24: Graphical Representation for Two-Way Action at Tower Leg

The loading around the perimeter is calculated using the results from a LARSA thick plate model, and summarized into shear stress plots around the critical section perimeter. Plots of this information are shown in the Figures 25 and 26 below.

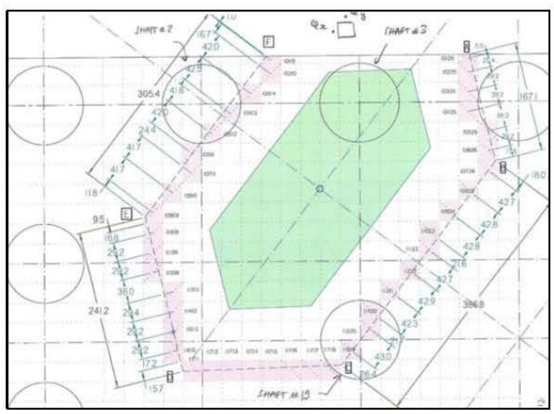


Figure 25: Summary of Plate Elements Considered for Shear Stress Along the Critical Section Perimeter

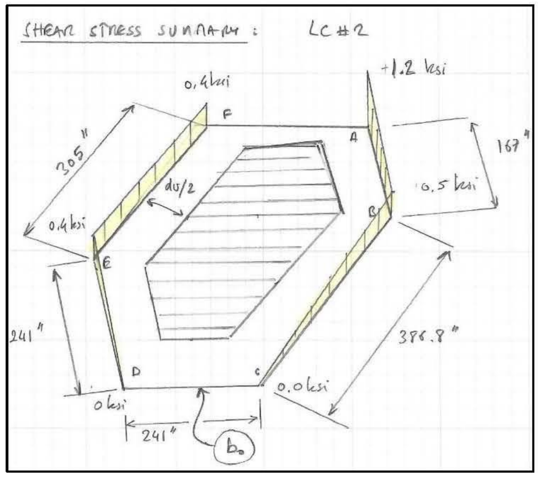


Figure 26: Shear Stresses at Critical Foundation Cap Sections Around the Tower Leg Perimeter under Maximum Loading – Side A-F Represents the Edge of the Cap

The loading pattern shows very high shear stress concentrations next to the edge of the foundation cap. The load resistance capability of the foundation cap is limited by having the leg so close to the edge. This prevents the distribution of the shear resistance to a “full” perimeter around the base of the leg.

Also, there is only one drilled shaft directly in the vicinity of tower leg, which forces this large concentrated loading to be redistributed through shear in the cap. In other words, if one were to assume that the drilled shaft directly beneath the toe of tower leg were infinitely stiff (i.e. a fixed vertical support), then all of the concentrated load from the tower leg would go directly to that shaft. However, that shaft is not infinitely stiff, and if it were infinitely stiff, it would not be able to resist all of the tower leg load by itself. Instead, it behaves as a spring (whose flexibility has been determined and considered herein), so the tower leg loading must be shared by the adjacent piles via the cap’s shear capacity, along with the cap’s bending capacity. As mentioned in §4.9.2.1 above, the cap’s transverse bending moment capacity at this location is insufficient, especially considering its lack of developed reinforcement. One-way action considered on a half-section was also found insufficient, as discussed above. ISA calculations for two-way action found equivalent D/C ratios up to 3.03, when checking the critical foundation cap section around the tower leg perimeter.

Because of the deficiencies noted above, a brick model of the foundation was created to validate the results from the thick plate model. The brick model included external loads from the tower legs, with reactions added to the piles. Plots are presented in Figures 27 and 28 below, showing the overall model and the external loading considered. The deformed shape with stress output presented in Figures 29 and 30 below, show close agreement with problematic localized behavior identified by the thick plate element model results presented above.

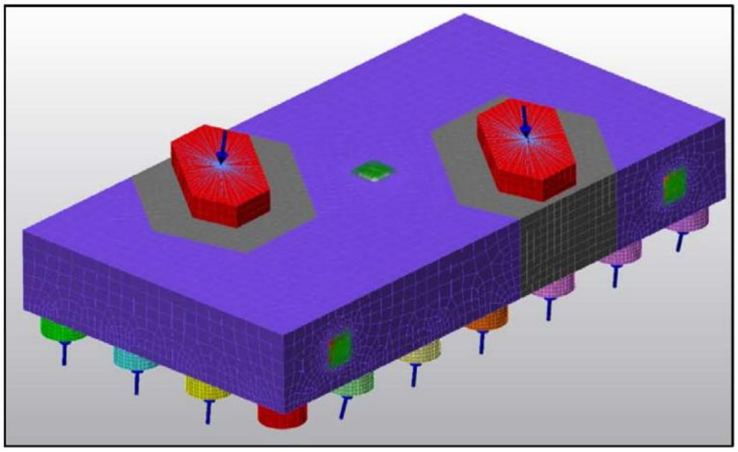


Figure 27: Brick Model Layout at Foundation Cap 1ST

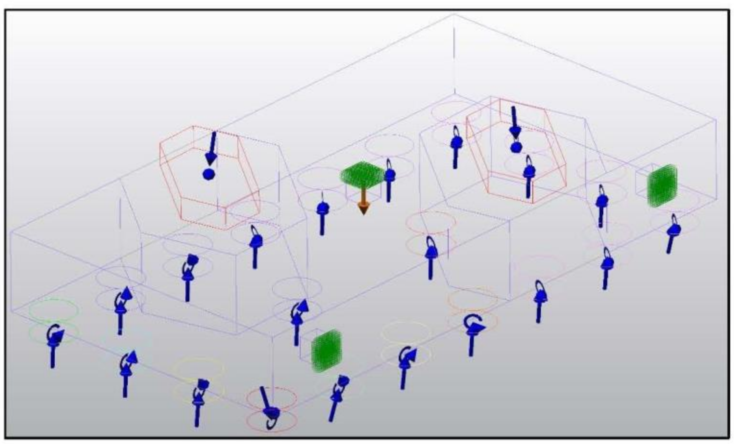


Figure 28: Brick Model External Loads at Foundation Cap 1ST

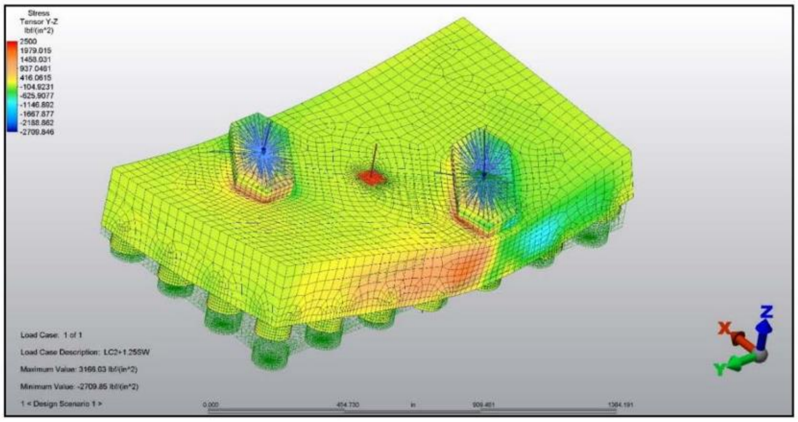


Figure 29: Brick Model Deformed Shape and Vertical Shear Stresses at Foundation Cap 1ST

The brick element model was able to provide the shear stress variation through the depth of the foundations cap. For comparison, the average shear stress over the depth of the cap at Corner A (the location of the greatest shear stresses) was computed to be 1.19 ksi, which closely agrees with the 1.20 ksi shear stress observed in the thick plate model at this location. Based on this, it can be concluded the thick plate model results were sufficient in identifying the deficiencies stated above. However, the brick element model revealed that the maximum shear stress at mid-depth of the foundation cap at Corner A exceeds 1.5 ksi, which is more than 25% greater than the average shear stress considered by the thick-plate element model used to identify the two-way action deficiency stated above.

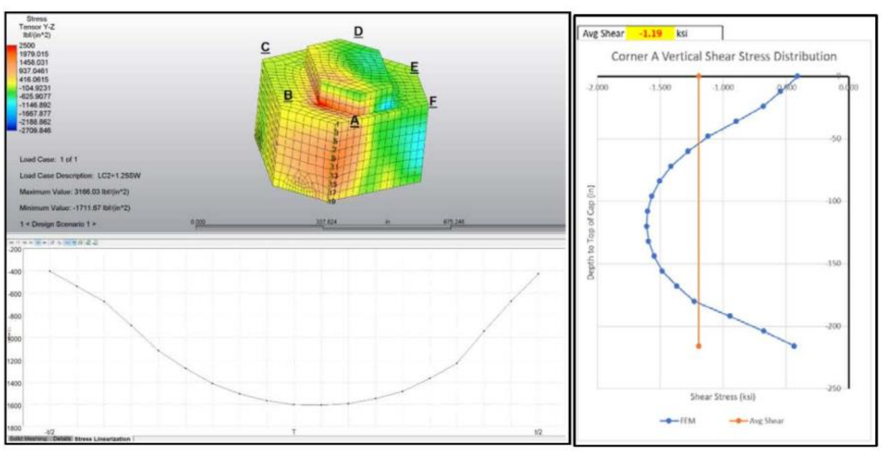


Figure 30: Brick Model Results at Two-Way Action Plane at Foundation Cap 1ST

4.9.2.3. Strut-and-Tie Models of the Foundation Cap

Various strut-and-tie models were also analyzed to determine if an alternate load path exists to carry the Extreme event tower leg loads to the drilled shafts through the foundation cap. The strut-and-ties models considered had demand-to-capacity ratios ranging from 2.0 to 2.7, and so a valid alternate load path was not found.

4.10. Upper Tower – Insufficient Capacity

The upper tower design has changed considerably, as the stay saddles have been replace with anchor boxes. ISA checks of the revised design are in progress.

4.11. Northbound Box Girder – Overcompressed Top Slab

Compressive stress limits are exceeded in the top slab of the northbound and southbound box girders near the pylon. The maximum top compressive stress is 5.5 ksi for both northbound and southbound girders, with a top slab slenderness factor ranging from 0.75 to 0.80 (ϕ_w). The AASHTO LRFD §5.9.4.2.1 (Table 5.9.4.2.1-1) compression limit of 0.60 x ϕ_w x f'_c is exceeded.

It is possible that the top slab compression and positive (sagging) moment could be reduced with a refined third-stage stressing. However, this is not provided in detail in the erection document 277609-NHB-MAN-MEM-01.

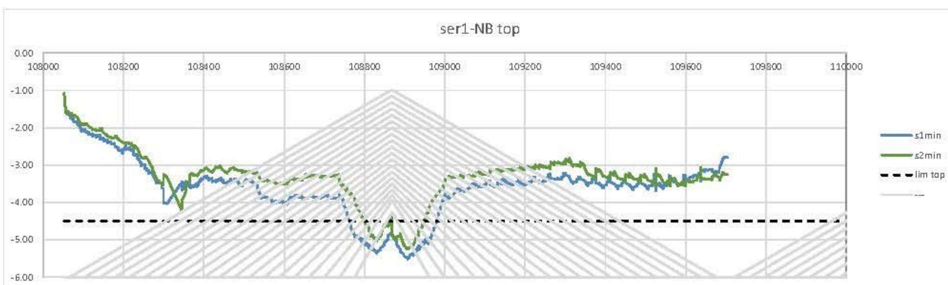


Figure 31: NB Service 1 top compressive stress

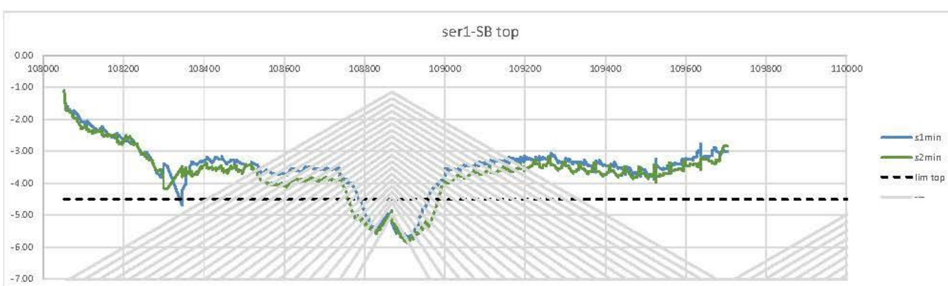


Figure 32: SB Service 1 top slab compressive stress

4.12. Box Girders – Insufficient Capacity

The box girder capacity is exceeded under the Strength loading combination for the segments adjacent to the pylon for both northbound and southbound box girders, with D/C ratios ranging from 1.2 to 1.5. This demand is results from maximum positive (sagging) moments, with the bottom fiber in tension.

It is possible that the sagging moment could be reduced with a refined third stage stressing. However, this is not provided in detail in the erection document 277609-NHB-MAN-MEM-01.

Another possibility is that the bottom post-tensioning could be increased in the segments adjacent to the pylon where the PT quantity seems inadequate. For example, there are only 5 - 1 ¼" PT bars in the bottom slab of the northbound box girder (similar for southbound).

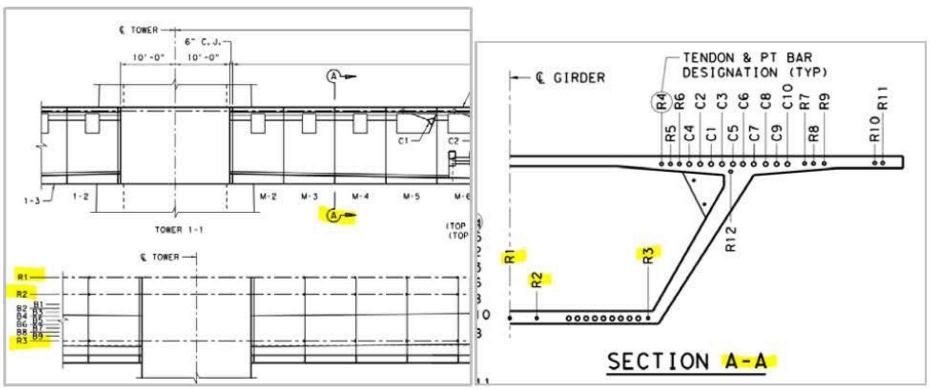


Figure 33: NB shown, 5 PT bars in bottom slab in the first 23 segments adjacent to the pylon – Schematic by Others

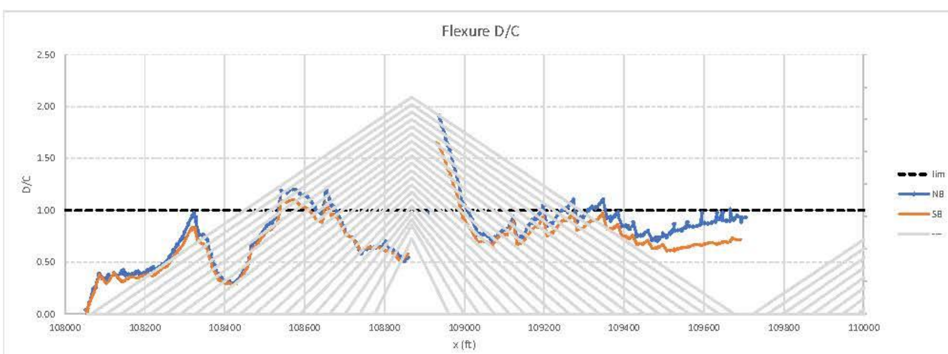


Figure 34: Flexural D/C ratio for NB and SB box girders

4.13. Box Girders – Mcr

In the original design, the minimum cracking moment (Mcr) was exceeded near midspan in the northbound box girder. This issue has been addressed by the updated design, since additional post-tensioning in the superstructure has been added. This issue may be considered resolved as a result of the design changes observed in Phase 2.

4.14. Delta Frames – Deficient Connection at Box Girders

The ISA team evaluated the demand on the connection between the delta frame and the precast block located at the bottom inside corner of the box girder and the cast-in-place grouted pour, shown as Section Σ1 in Figure 35 below. The shaded region, critical to the transfer of loading from the box girder to the delta frame and supporting stay cables, is unreinforced, with the exception of the two tendons located roughly at the center of its section. ISA model results indicate that this section is subjected to both axial and bending demand, leading to tension at the top fiber of the precast block and the unreinforced region of the connection. Moreover, this joint must transfer shear across the unreinforced joint.

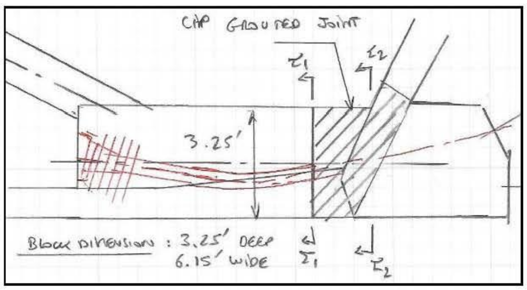


Figure 35: Sketch of bottom block connection between the delta frame and box girder

The top fiber of the unreinforced cast-in-place (CIP) grouted connection between the base of the box girders (both Northbound and Southbound) and the bottom node of the delta frame is subjected to tension under both unfactored permanent loading and the Service I load combination, exceeding the no-tension requirement for joints without bonded reinforcement mandated by AASHTO LRFD §5.9.4.2.2.

The grouted connection should be redesigned, with the objective of introducing longitudinal and transverse reinforcement across the entire length of the CIP grouted connection. The additional reinforcement should be made continuous using doweled reinforcement and couplers, so that the reinforcement extends across interfaces between CIP pour-back concrete and adjacent precast concrete elements.

4.15. Delta Frames – Anchor Head Shear Friction Joint

The anchorage and the local confinement reinforcement (spirals) of the tendons TD1S and TD4 are located next to the delta frame shear key. This is a very unusual and unconventional detail, as the anchorages create a highly disturbed state of stress at a critical interface. The shear key transfer plane is located within the local zone of the PT anchorage, as defined by AASHTO LRFD §5.10.9.2.3, where large localized tensile stresses are expected and where reinforcement congestion is considerable, as shown in Figure 36 below. The reduction in this shear plane's capacity due to the presence of these tendon anchorages is unknown, since no provision exists within AASHTO LRFD to account for this configuration. As such, the ISA team cannot validate that the shear key as designed meets the AASHTO LRFD specifications.

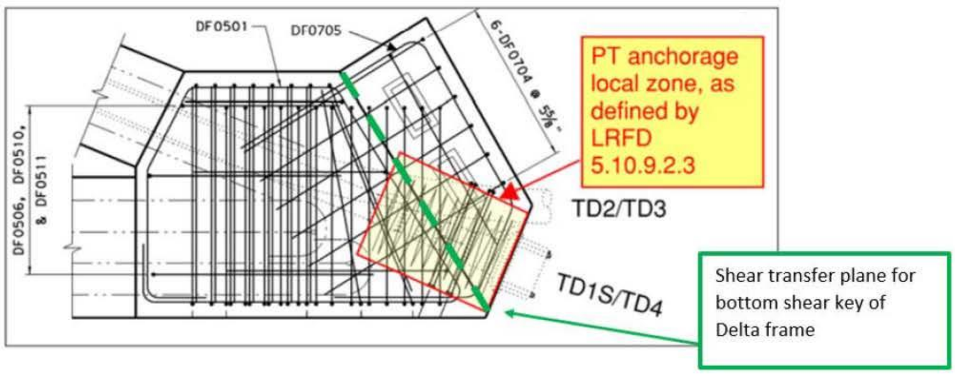


Figure 36: shear key reinforcement and location of tendon local anchorage zone

4.16. Delta Frames – Overstressed Diagonal Strut

The maximum tension stress in top and bottom fiber of the diagonal struts under Service loading combinations exceeds the stress limitation requirement of AASHTO LRFD §5.9.4.2.2.

4.17. Delta Frames – Insufficient PT Bursting Reinforcement

The vertical bursting reinforcement provided behind the anchors of tendons TD2 and TD3 in the bottom block of the precast box girder which is connected to the delta frame is not sufficient when the Tendons TD2 and TD3 are composed of Ø0.62" strands, which is the case for the back span stay cables 16 to 19, per drawing NHB198D.

4.18. Delta Frames – Bottom Strut Insufficient Capacity

The ultimate capacity of the bottom strut of delta frame Type 1 is exceeded under the governing tensile demand resulting from the Extreme III - loss of stay combination, when the loss of two side-by-side stays are considered (example N19M-SB and N19M-NB). This conclusion remains true whether the losses of the two adjacent stay cables are considered to occur simultaneously or sequentially.

4.19. Inclined Stay Cables – Insufficient Capacity

The loads in the inclined stay cables are exceeding capacity at multiple locations, considering load factors according to AASHTO LRFD §3.4.1 (per PTI DC45.1-12 §5.3) and resistance factors according to PTI DC45.1-12 §5.3.3.

There are discrepancies between the stay sizes shown in the Erection Manual vs. those shown on the drawings. The stay force data tables from the original design have been voided in the drawing package M13A-01; however, they have not been replaced with updated tables.

4.19.1. Considering Stay Data from the Design Drawings

The number of deficient stays (i.e. D/C ratio >1) under Strength load combinations excluding Strength IV is 21 out of 152 total stays. The D/C ratios are as large as 1.12.

The number of deficient stays including Strength IV is 41 out of 152 total stays, with D/C ratios as large as 1.16.

4.19.2. Considering Stay Data from the Erection Manual

The number of deficient stays (i.e. D/C ratio >1) under Strength load combinations exluding Strength IV is 8 out of 152 total stays. The D/C ratios are as large as 1.12.

The number of deficient stays including Strength IV is 21 out of 152 total stays, with D/C ratios as large as 1.16.

4.20. Bearings – Insufficient Uplift Capacity

The bearings are not able to resist the uplift loading condition observed. The overall system of bearings and vertical stays are incompatible under this loading condition. Significant uplifts at the bearings at Piers 2N and 2S have been observed at the Service and the Strength limit. These uplift values are not compatible with the connection system composed of the bearing and the vertical tie down (or vertical stay cables). Bearing uplift is prohibited by the project requirements under Service loadings. Under Strength loadings, the current detail would create a significant risk of pounding and vertical relative displacement between the deck and the substructure. An increase in the size of the tie down and/or modification of the bearing detail is likely required.

In addition, there are discrepancies in the permanent bearing reactions shown in the Erection Manual versus those shown on the contract drawings. The Service bearing reactions on sheet NHS 124 can be shown to produce uplift by substituting the permanent loads from the Erection Manual for the permanent loads provided in the bearing design table shown on the drawings. The table below presents this substitution, and reveals uplift (negative values shown in red indicate uplift):

	NB Box Line Service Bearing Reactions					
Pier	2N-NB	2N-NB	1N-NB	1S-NB	2S-NB	2S-NB
Location	Ext.	Int.	Center	Center	Ext.	Int.
Vertical Min Service Contract Drawing (NHB 124)	800	5	1500	1500	900	5
Permanent Contract Drawings (NHB 124)	1800	1500	4000	4100	1900	1300
Permanent Erection Manual (sheet 169 of 170)	2296	859	4074	4164	1737	329
Vertical Min Service (Sub Erection Manual Perm.)	1296	-636	1574	1564	737	-966

	SB Box Line Service Bearing Reactions					
Pier	2N-SB	2N-SB	1N-SB	1S-SB	2S-SB	2S-SB
Location	Int.	Ext.	Center	Center	Int.	Ext.
Vertical Min Service Contract Drawing (NHB 124)	400	1300	1700	1800	500	1200
Permanent Contract Drawings (NHB 124)	1700	2300	4100	4300	1700	2300
Permanent Erection Manual (sheet 169 of 170)	1184	2466	4190	6570	178	2033
Vertical Min Service (Sub Erection Manual Perm.)	-116	1466	1790	4070	-1022	933

"-" Indicates Uplift

The ISA independent calculations are not matching precisely with either the drawings or the Erection Manual. However, uplift was observed under both Service and Strength load combinations in the ISA calculations.

4.21. Vertical Stay Tie-Downs – Insufficient Uplift Capacity

The vertical stays do not have adequate capacity to resist the uplift forces once the bearings decompress, and have deficient redundancy for the loss-of-stay case. For Service loadings the interior bearings at 2N and 2S have insufficient pre-compression from the vertical stays to resist the uplift force for the northbound and southbound box girders. For Strength loadings, the interior and exterior bearings at 2N and 2S and additionally the bearings at 1N and 1S have insufficient pre-compression from the vertical stays to resist the uplift force for the northbound and southbound box girders.

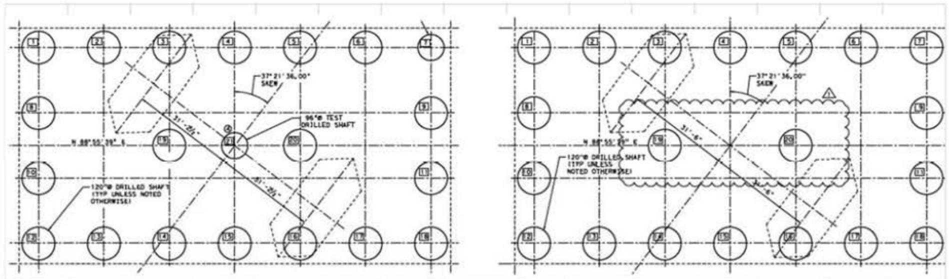
Since unrestrained bearing uplift is not allowed by AASHTO LRFD §14.6.1, the ISA 3D construction model was not intended to directly capture the increase in vertical stay force at the onset of bearing lift-off. An incremental post-processing routine has been conducted to estimate the increase in vertical stay force demand after lift-off. These results indicate a D/C ratio > 1.0 for the vertical stays under Strength loadings.

4.22. Tower Foundations – Settlement

The reported settlements in the project Geotechnical Report are large. The ISA Team performed an independent review and found an appreciable difference (lower values). The settlement values are large with potential side effects and necessary considerations (e.g. vertical camber of pylon).

4.23. Tower Drilled Shafts – Tension Not Allowed

The TxDOT Bridge Design Manual - LRFD (BDM LRFD Ch 4 §2) does not permit drilled shafts to be in tension under the Service I load combination. Drilled shafts beneath each of the towers experience tension when considering the Service I load combination, as shown in Figure 37 below.



1NT 1ST

CASE 0: MAX TENSION IN ANY DRILLED SHAFT (NON-CONCURRENT RESULTS), KIPS														
-231	-651	-1,033	-899	-425	6	778		-317	-669	-1,056	-949	-494	-238	-85
-453						-9		-313						-223
		-1,645		-1,548						-1,176		-1,194		
-88						-378		-175						-329
141	-124	-658	-1,198	-1,270	-799	-274		730	-171	-320	-601	-755	-534	-288
1NT								1ST						

Figure 37: Summary of Service I Tension Loads for 1NT/1ST Drilled Shafts – Drilled Shaft Layout Schematic by Others

In BDM LRFD Ch 4 §2, TxDOT permits discretionary exceptions to this requirement. However, there are no such exceptions to AASHTO LRFD §5.7.4.5 for Finding 07, which is related to this observation.

4.24. Tower Drilled Shafts – Extents of Longitudinal Reinforcement

Verification of the extents of the longitudinal reinforcement in the drilled shafts is performed in accordance with the TxDOT Geotechnical Manual and Common Foundation Details (FD) drawing.

The TxDOT Geotechnical Manual (2018) Chapter 5 §3 requires that the longitudinal reinforcement extend the full depth of the shaft.

Additionally, the TxDOT Common Foundation Details (FD) drawing (fdstde01-20.pdf) shows typical longitudinal reinforcement extending the full depth of the shaft.

The design drawings show 30-#11 bars extending to the bottom of all drilled shafts. However, the longitudinal reinforcement ratio for these bars is only 0.41% for drilled shaft Types A-F, and 0.65% for

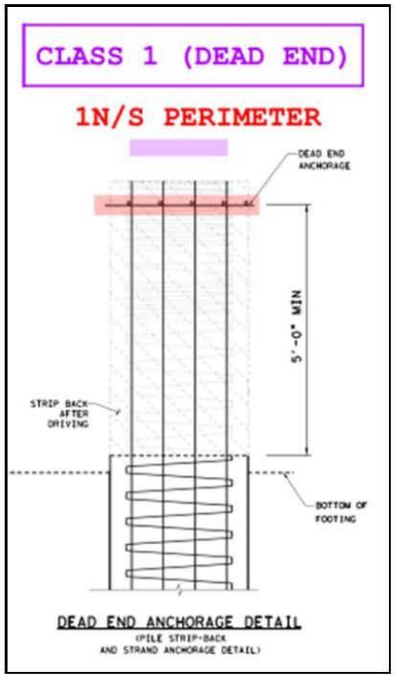
Type G. Both of these ratios are smaller than AASHTO and TxDOT requirements (refer to Finding 8). Additionally, notes on the design drawings indicate that the longitudinal reinforcement in the bottom portion of the shaft is intended for "the means and methods of construction…".

In comparing the TxDOT requirements with the details shown on the design drawings, it appears that the drilled shaft longitudinal reinforcement does not extend far enough into the lower portion of the shafts.

4.25. Backspan Piers Foundation – Vague Strand Anchorage Details

The driven piles at Pier Foundations 1N and 1S are connected to the pile cap using a modification of the "strip-back detail" shown on the TxDOT Prestressed Concrete Piling (CP) drawing. The upper 60 inches of the strands that were initially bonded in the pretensioned driven piles are exposed by this "strip-back" procedure.

For the "Class 1, Type Dead End" piles located around the perimeter of the foundation, the original design and the current drawings specify a dead-end anchor for unstressed strand. The drawings state that the details should be approved by the Engineer. It is not clear if "Engineer" refers to TxDOT or the Engineer-of-Record (EoR) or both. The details remain vague and are not specified. The design check of the piles under tension (if tension is permitted, see related Finding 26) is dependent upon the anchorage and development of the passive strands. The design is incomplete and cannot be verified without these details.



⑧ CONTRACTOR TO SUBMIT DEAD END ANCHORAGE DETAILS TO ENGINEER FOR REVIEW AND APPROVAL. THE DEAD END ANCHOR MUST DEVELOP THE FULL TENSILE CAPACITY OF EACH INDIVIDUAL STRAND.

Figure 38: Vague Dead End Strand Anchorage Detail in Pier Foundation 1N and 1S – Detail Provided by Others

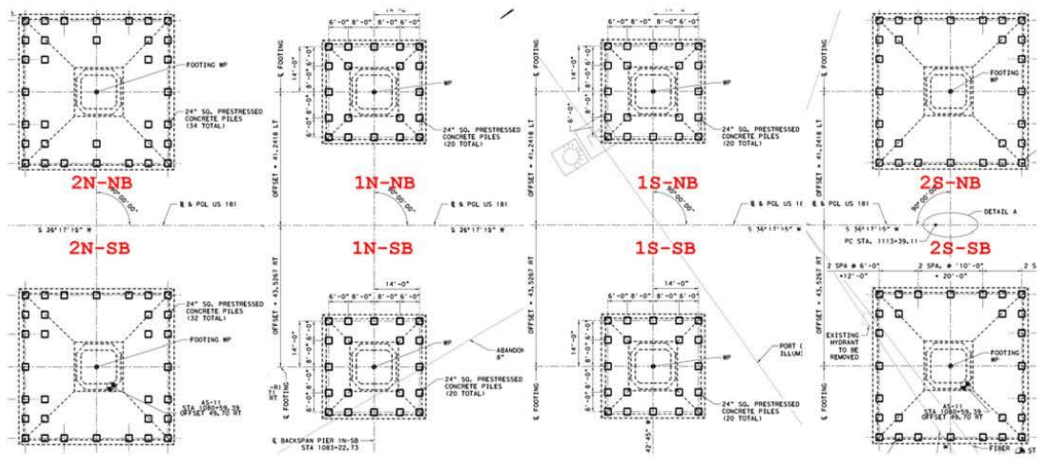
4.26. Backspan Piers Driven Piles – Tension Not Allowed

Driven piles in the backspan pier foundations were found to have tension/uplift forces under Service I load combinations. However, the TxDOT Bridge Design Manual Chapter 4 §2 does not permit this condition without approval.

The maximum axial tension demand in the Service I limit state is provided below for the driven piles at backspan Piers 1N/S and transition Piers 2N/S:

	Tension (kips) -- SERVICE I		
Fndn	Demand	Allowable	Tension?
2N-NB	-12	0	NO
2N-SB	-16	0	NO
1N-NB	199	0	YES
1N-SB	181	0	YES
1S-NB	199	0	YES
1S-SB	168	0	YES
2S-NB	-13	0	NO
2S-SB	-19	0	NO

The maximum tension in each of the driven piles under the Service I load combination is presented below for backspan Piers 1N/S and transition Piers 2N/S:



CASE 0: MAX TENSION IN ANY DRIVEN PILE (NON-CONCURRENT RESULTS), KIPS

-13	-16	-19	-23	-21	-17	-12														-15	-19	-24	-24	-20	-16	-13
-16	-19		-26		-19	-14		199	149	80	2	6		118	55	76	148	199		-17	-22				-20	-16
-19	-22				-21	-16		172	123		-7	16		99	32		122	173		-20						-20
-22						-19		133				30		89				135		-22						-23
-23	-26				-25	-20		99	44		-6	42		69	-1		40	99		-23						-24
-23	-26		-34		-25	-20		71	15	-9	5	63		53	-4	-8	10	69		-23	-28				-28	-24
-21	-26	-29	-31	-26	-22	-18														-22	-26	-30	-35	-32	-28	-24
2N-NB								1N-NB						1S-NB						2S-NB						
-21	-24	-26	-27	-25	-23	-20														-23	-27	-29	-31	-30	-27	-23
-22	-25				-26	-21		91	29	-8	25	81		84	25	-7	21	80		-24	-29				-27	-24
-22	-25				-26	-21		101	42		-4	44		108	42		42	100		-25						-25
-22						-21		137				39		136				123		-25						-25
-20	-23				-24	-20		160	109		24	86		150	88		97	149		-23						-23
-18	-21				-22	-18		181	132	63	60	120		158	95	43	115	168		-21	-26				-24	-21
-16	-19	-22	-25	-25	-21	-17														-20	-24	-29	-29	-25	-22	-19
2N-SB								1N-SB						1S-SB						2S-SB						

Figure 39: Backspan Piers with Tension in Piles – Pile Layout Schematic Provided by Others

For backspan Piers 1N/S, the driven piles are subject to tension under the Service I load combination.

In BDM LRFD Ch 4 §2, TxDOT permits discretionary exceptions to this requirement.

4.27. Backspan Piers Foundation – Passive Strand Development

The driven piles at Pier Foundations 1N, 1S, 2N, and 2S are connected to the pile cap using a modification of the "strip-back detail" shown on the TxDOT Prestressed Concrete Piling (CP) drawing. The upper 60 inches of the strands that were initially bonded in the pretensioned driven piles are exposed by this "strip-back" procedure.

For the "Class 1, Type II" and "Class 2 Type II" piles, the original design and the current drawings specify a passive strand development length of 60 inches, extending above the top of the stripped back pile. Calculations for passive strand development into the pile cap are not specified by AASHTO LRFD, and very little research on this topic exists. The design check of piles under tension (if tension is permitted, see related Finding #26) is dependent on the development of these passive strand, and therefore cannot be verified.

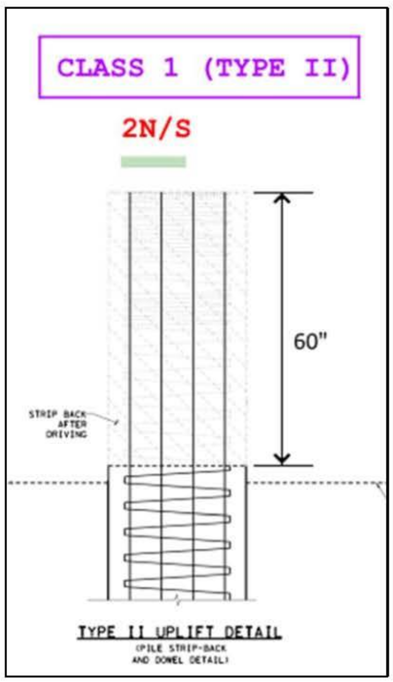


Figure 40: Passive Strand Development in Pier Foundation 1N, 1S, 2N, and 2S – *Detail Provided by Others*

4.28. Backspan Piers Foundation Cap –Reinforcement Detailing Issues

The reinforcement details in the Pier 1N, 1S, 2N, and 2S pile caps are unconventional for load transfer where the piles are in significant tension. The load transfer mechanisms from the pile cap to the piles is indirect, and it is not detailed with anchorage lengths typically used for piles in tension. These details have not been updated in the current design. Findings 25 and 27 cover the lack of design and AASHTO LRFD information related to the development and anchorage of passive strands in the pile cap. Besides these strand issues, uplift demand from piles are potentially carried by vertical reinforcement bars surrounding each top of pile and by dowel bars included in the design. However, these reinforcement bars have detailing and strength issues that compromise their ability to carry vertical tie forces, as follows:

- In Pier 1N and 1S, the six vertical MF0804 reinforcement bars surrounding the interior piles have insufficient tie capacity with D/C = 1.27.
- In Pier 1N and 1S, the eight vertical MF0803/MF0804 reinforcement bars surrounding the perimeter corner piles have insufficient tie capacity with D/C = 1.28.
- In Pier 1N and 1S, the top hooks of the vertical MF0803 and MF0804 reinforcement bars terminate below the top mats of reinforcement. Typically, vertical tie bars are hooked above the top mat to ensure the integrity of the top nodes of strut-and-tie models.
- In Pier 1N and 1S, the #6 dowel bars are post-installed with adhesive into the stripped back pile. Due to the proximity of these bars to the pile face, the expectation of cracked concrete, and the dowel spacing, the resistance of these dowels is significantly less than that of cast-in reinforcement bars. Any contribution of these dowel bars to transferring uplift pile capacity is insignificant and should be neglected.

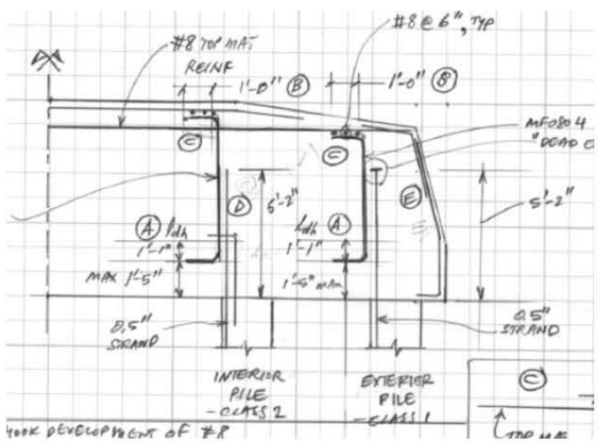


Figure 41: Indirect Load Path of Vertical Ties Transferring Pile Tension

Various Pier 1N, 1S, 2N, and 2S pile cap details do not meet AASHTO LRFD requirements, as listed below. These details have not been updated in the current design.

- In Piers 1N, 1S, 2N, and 2S, the pile cap vertical reinforcement bars do not satisfy transverse reinforcement detailing requirements, per AASHTO LRFD §5.6.3.4.2 and §5.11.2.6.1, and hence cannot act effectively as shear reinforcement or as ties in strut-in-tie model load paths. Extending these bars such that they are as close to the surfaces of members as cover requirements permit and hooking above the top mat and below the bottom mat ensure that a complete and continuous load path is achieved.

- In Piers 1N, 1S, 2N, and 2S, the pile embedment of 2 inches (after strip back) is less than the minimum required embedment of 6 inches, per AASHTO LRFD §10.7.1.2.
- In Piers 1N, 1S, 2N, and 2S, the quantity of steel required for shrinkage and temperature reinforcement along the exposed side faces of the pile cap, per AASHTO LRFD §5.10.8, is greater than the quantity of vertical surface steel provided.
- In Piers 2N and 2S, the interface shear steel provided that crosses the construction joint at 3'-0" above of the bottom of pile cap does not meet the minimum area required per AASHTO LRFD §5.8.4.4.
- In Piers 2N and 2S, the diameter of the dowel reinforcement exceeds the diameter of corresponding column longitudinal reinforcement by more than 0.15 inches, which is not permitted per AASHTO LRFD §5.13.3.8. Additionally, the #14 outer face dowel reinforcement bar is larger than maximum size permitted of #11.
- In Piers 1N, 1S, 2N, and 2S, the pile cap bottom mat reinforcement bars exceed the maximum spacing requirement of 18 inches in the zones between piles, per AASHTO LRFD §5.10.3.2; see Figure 42 below.

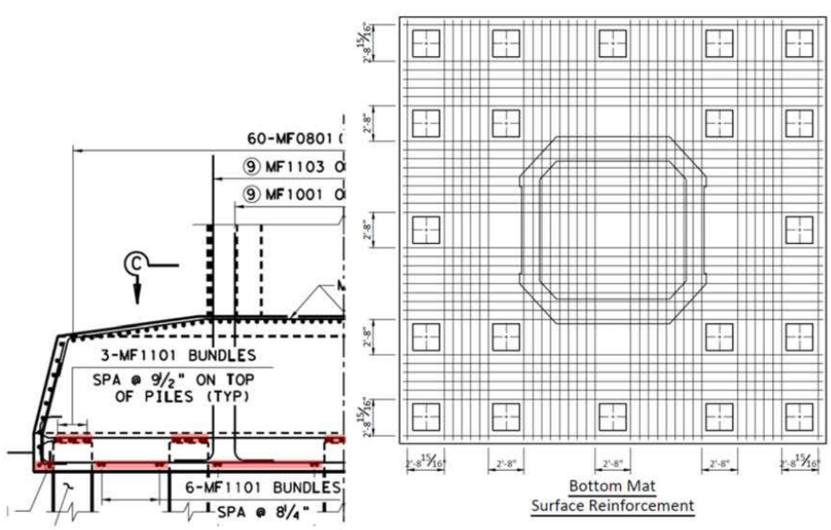


Figure 42: Gaps in Pile Cap Bottom Surface Reinforcement in Between Piles

4.29. Tower – Insufficient Flexural Capacity

While ISA checks of the revised design are still in progress, it has been found that Lift 17 in the updated design has a D/C ratio = 1.02. This is the second lift in the upper pylon in which the concrete strength switches from 10 ksi to 8 ksi. The D/C ratio could possibly be reduced (and this issue resolved) by changing the extents of upper tower concrete to 10 ksi. This may have been intended in the design; however, the General Notes drawing NHB 0B still calls for 8 ksi for this lift.

4.30. Tower – Insufficient Shear Capacity

This issue may have been resolved with the revised upper tower design.

ISA checks of the revised design are in progress.

4.31. Tower – Nodal Zone Detailing

The reinforcement detailing within the nodal zone does not have clear load paths for the transfer of load, and these details have not been significantly updated in the current design. However, the concrete strength within the nodal zone has been increased.

In Phase 2, the ISA team were able to identify satisfactory load paths to carry the design forces, considering the increase of concrete strength in the nodal zone. This issue may be considered resolved as a result of the design changes observed in Phase 2.

4.32. Superstructure – Vibration

The ISA team performed vibration checks indicating that the vertical acceleration exceeds the project limit of 0.05g per TP §13.2.1.12. The Wind Report §7.1.3 also acknowledges that this limit is exceeded. This check is best conducted as a full assessment of the dynamic response of the deck under vehicle, pedestrian and wind loading by the wind consultant. The detailed results of wind time-history analyses performed by the wind consultant should be provided by the Design-Build Team's wind expert in order to confirm if the vertical deck vibration remains within the 0.05g limitation for a wind speed of 30 mph.

4.33. Superstructure – Top Slab Transverse Overstress

The bottom fiber stresses in the southbound box girder at the top slab haunch exceed the allowable stress limits of AASHTO LRFD Table 5.9.4.2.2-1 under Service load combinations, as show in Figure 43 below.

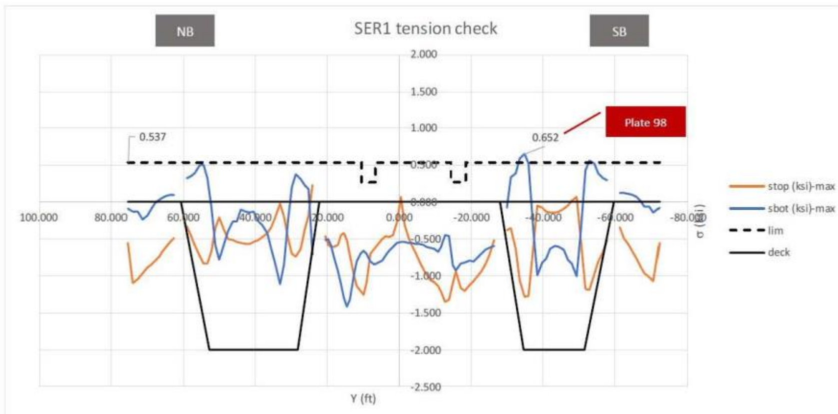


Figure 4-1:

Figure 43: Top slab Service tension check

4.34. Delta Frames – Insufficient Details

Multiple observations were made related to the delta frame details in the Phase 1 report.

This item involved several discrepancies, missing dimensions, concrete strengths, reinforcement details, grout details, and specifications, etc. Some of these elements have now been provided while some other are still missing.

- The dimensions and reinforcement details of the CIP central deck top slab were omitted from the original drawing packages but are now included in the revised drawing set provided. This issue may be considered resolved.
- Discrepancies between the concrete strengths specified in the General Notes drawings and the delta frame drawings NHB 194 have now been resolved. Concrete strength for the precast delta frame element is now consistently defined as 10 ksi. This issue may be considered resolved.
- No material properties are provided on the drawings regarding the CIP grout between the bottom blocks and the delta frame shear key. A strength f'c = 8 ksi has been assumed for the ISA checks. It is also assumed that aggregate will be provided in accordance with the grout manufacturer's recommendation in order for the grout-to-concrete joint to be able to carry shear via interface shear transfer. Given the significant shear and bending demand on this joint, its critical nature for the structure, and the small space in which the material needs to be injected, a precise definition of the material to be used needs to be provided. This missing information is not shown in the revised drawings, and therefore this issue remains unresolved.

4.35. Vertical Stay Tie-Downs – Not Allowed by Technical Provisions (ATC)

Section 13.2.1 of the Technical Provisions does not allow permanent tie-downs; however, an ATC from the Design-Build team was permitted. The ISA team has not reviewed the conditional requirements that may be associated with this ATC.

4.36. Vertical Stay Tie-Downs – Insufficient Lower Anchorages

The drawing details show that the lower anchorages of the vertical stays resemble post-tensioning anchors instead of stay anchors.

The drawings in package MSUB-B Rev00 retain the details of the original design. The tie-downs behave as stays, and their detailing must accommodate the expected longitudinal displacements of the deck relative to the piers.

New Findings (Since Phase 1):

4.37. Box Girders – Torsional Cracking During Erection

The torsion resulting from unbalanced dead load and equipment loadings presented in the Erection Manual (277609-NHB-MAN-MEM-01) is in the range of 50,000-80,000 kip-ft.

This unfactored demand, without consideration for wind, is greater than the torsional cracking moment T_{cr} (AASHTO LRFD §5.8.6.3).

A beam element model tends to overlook certain effects, especially for a bridge of this width with stays anchored at the middle of the superstructure between the box girders. When considering the effects of shear lag of the horizontal components of the stay forces using a plate element model, the ISA team has found that principal tensile stresses in the webs exceed the allowable values specified by AASHTO LRFD §5.14.2.3.3. Figures 44 - 46 present an example of this problem at Segment 7 when stressing Stays 26, where principal tensile stresses in the webs exceed that allowed by AASHTO LRFD.

Figure 44: Bottom 3D rendered view of the plate model

When looking at the compressive incremental stresses in top slab due to Stay S26 stressing, Figure 45 below shows that diffusion is only complete at approximately 70 ft in front of the stay forces.

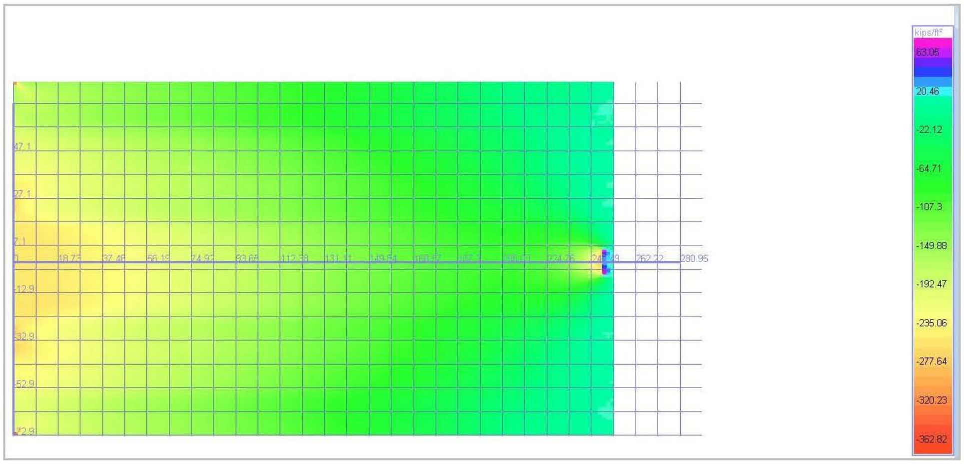


Figure 45: Top slab incremental stresses for stay stressing S26 (the grid is 10 ft by 10 ft)

The left-hand graph in Figure 46 below shows principal tensile stresses in the webs for the northbound box girder and the right-hand graph shows principal tensile stresses in the webs for the southbound box girder.

The axis z=0 ft corresponds to the top of the web (either the outer or inner web). Note that the analysis presented below was carried out previously with derrick crane loads that are lighter than the current ones presented in the updated Erection Manual. Therefore, the principal tensile stresses for the current derrick crane loads would actually be worse than those presented below.

Northbound Box Girder Webs **Southbound Box Girder Webs**

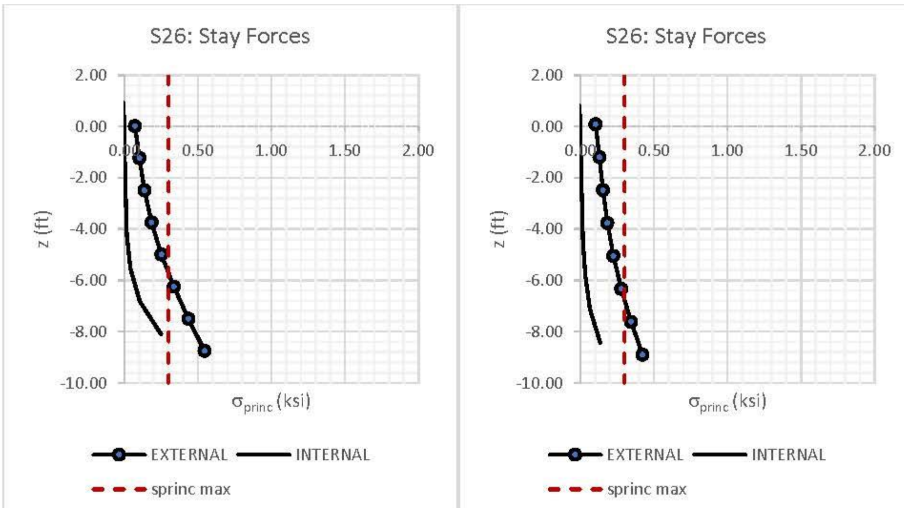


Figure 46: Web principal tensile stresses at Segment 7 when stressing the S26 stay cables

4.38. Delta Frames – Vertical Brace Insufficient Capacity and Detailing

The delta frame vertical bracing element has insufficient flexural capacity to resist the loss of one stay of a side-by-side pair of stays. For the loss of stay N19M-NB, the ISA found a D/C ratio = 1.17 considering a fracture dynamic force (FDF) equal to 2.0. PTI DC45.1-12 §5.5 allows for a reduction of the FDF as determined by nonlinear dynamic analysis of a sudden cable rupture; however, in no case shall the FDF be less than 1.5 times the static force in the cable. In order to investigate the potential reduction in FDF, the ISA team will perform nonlinear dynamic analyses and report its findings in the Phase 2 Part 2 report.

4.39. Wind Report

The revised Wind Report is considered deficient due to the following issues:

- The evaluation of stay cable vibration was performed on the incorrect/inconsistent cable sizes in the report, which do not match the final cable sizes on the drawings. There are discrepancies between stay pipe diameters, number of cables per stay, and stay force.
- The report is missing key data that is necessary for the design of the bridge. The Technical Provisions §13.2.1.7 require documenting the specific wind study results in the report; however, per Appendix E, Table 11, this data is unavailable due to its size. This omits important information that cannot be independently verified or controlled for quality assurance.
- The cable vibrations are evaluated under unrealistically low wind speeds, and the consideration if supplemental damping is required is incomplete.

- The Technical Provisions §13.2.1.7.3 require a "full aeroelastic model … including approach spans adjacent to each end of the New Harbor Bridge." Per photos in the report (see Figure 4 for example), it appears that no approach spans were included in the aeroelastic model, and therefore the model is not compliant.

4.40. General Notes

The General Notes in drawing package 277609-NHB-PLN-M02A-00 have been updated, and the ISA team questions the following:

1. General Notes III, note 10.0.A

   On previous revisions of this drawing, this note stated: "The minimum concrete strength at time of releasing forms for precast segments shall be as shown in the plans."

   This note has been removed from the drawing, and it now states "Not used."

   The minimum concrete strength at the release of the forms should be provided on the drawings.

2. General Notes III, note 10.0.D

   On previous revisions of this drawing, this note stated: "The minimum age of precast segments when first erected shall not be less than that shown in the plans."

   This note has been revised, and now states, "The minimum age of precast segments when first erected shall not be less than that shown in the erection manual."

   The minimum age of precast segments when first erected should be provided on the drawings.

3. General Notes IV, note 18.0

   This note has been added to the drawing. Point D of the note indicates that the geometry shown on the plans corresponds to a settlement of 14" at the towers.

   As discussed in Finding 22, the 14" settlement at the towers has not been observed by the ISA geotechnical analysis. As such, it would be prudent to develop a contingency plan regarding the final geometry of the structure if the actual settlement value is different from the 14" currently estimated.

4. General Notes IV, note 19.0

   This note has been added to the drawing. The note discusses future deepening of the channel.

   Plans for the future deepening of the channel have not been considered in the ISA assessment. Additional analysis would be required by the ISA team to evaluate this note.

4.41. Geometry

The Erection Manual (document 277609-NHB-MAN-MEM) presents a detailed construction sequence that was implemented in the ISA staged construction model. The sequence provided was used to tune the stays along with the incremental changes of deflection. However, cambers are not provided. It is understood that the deflections values provided include corrections that are made during the erection, most probably when setting the first segments after the cast-in-place joints. These joints are located between segments 14 and 15, 26 and 27, 42 and 43, 54 and 55, 70 and 71, 82 and 83. These joints are then spaced by stretches of 12 or 16 segments. Even if corrections are applied at CIP joints, the differential deflections that occur during the erection of a 12 or 16 segments stretch are substantial and may generate difficulties, especially when erecting the three or four delta-frames during that stretch of erection.

As an example for the stretch between segments 27 to 42, see Figure 47 below:

The differential deflection of NB, when erecting segment 42 is: $\delta_{NB}[42] = D_{NB}[42] - D_{NB}[26]$

The differential deflection of SB, when erecting segment 42 is: $\delta_{SB}[42] = D_{SB}[42] - D_{SB}[26]$

Then the total differential deflection within this stretch is: $\boldsymbol{\Delta_{diff}[42] = \delta_{NB}[42] - \delta_{SB}[42]}$

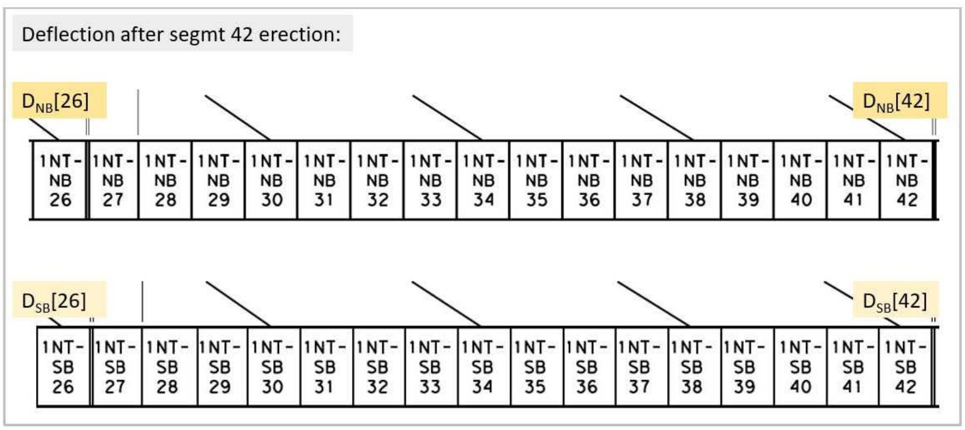


Figure 47: Example of differential deflection check during one stretch of erection – schematic by others

These differential deflection computations were carried out for the following stretches with the following results:

Stretches segment XX-YY	Δ_{vert} (in)	Δ_{hz} (in)
27-42	3.2	1.2
43-56	3.3	1.4
57-70	6.2	2.4
71-82	5.9	2.0

For instance, the vertical differential deflection between both decks, for erection of stretch 71 to 82, is calculated to be 5.9 in. The transverse differential displacement is calculated to be 2.0 in for that same stretch of segments. These are significant differential displacements that will be difficult to control solely with shims within allowable tolerance. These displacements are likely to increase if the box girders crack (see Finding 37).

4.42. Back Span Piers – Column Reinforcement Details

The following column reinforcement details do not meet the requirements for hollow rectangular compression members:

- In Piers 1N and 1S, the ratios of reinforcement areas between outer and inner faces of a given column wall are 2.46 (bundled #11 to #10), 1.57 (#11 to #9), and 1.23 (#11 to #10), which are not permitted. In Piers 2N and 2S, the maximum ratio of reinforcement areas between outer and inner faces of a given column wall is 3.75 (bundled #14 to #7), which is not permitted. Per AASHTO LRFD §5.10.12.1, the two reinforcement areas shall be approximately equal.
- In Piers 1N, 1S, 2N and 2S, the cross ties in the walls of the hollow compression members do not meet the requirements of AASHTO LRFD §5.10.12.3, because (1) the cross ties are not shown to enclose both lateral and longitudinal bars, and (2) cross ties are not staggered to restrain each lateral and longitudinal bar.

EXHIBIT B
SUMMARY OF SELECTED IBT FINDINGS AND CDA PROVISIONS[2]

IBT Finding	Requirements Not Met
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
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 strength 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.
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.
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.)

[2] Reference is made to the full IBT Report 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.

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.
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
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.
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
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.
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
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)

[6] Incorporated by Section 13.1 of the TPs.

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
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)
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.
Stay cable loads exceed capacity at 101 out of 152 stays. The demand/capacity ratio is greater than 1.0	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 Section 3.2.1 (all Design Work and Construction Work shall be in accordance with Good Industry Practice) PTI DC45.112 §§ 5.3.
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
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.
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

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
There are at least four significant findings related to the Wind Report.	TP Section 13.2.117
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
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
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