INDEPENDENT STRUCTURAL ANALYSIS FOR THE CORPUS CHRISTI NEW HARBOR BRIDGE PROJECT

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TECHNICAL MEMORANDUM

TOWER DRILLED SHAFTS

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DOCUMENT NUMBER: TM1001

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Revision History

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1. Introduction

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

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

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

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

2. References

The following documents are referenced in this memorandum.

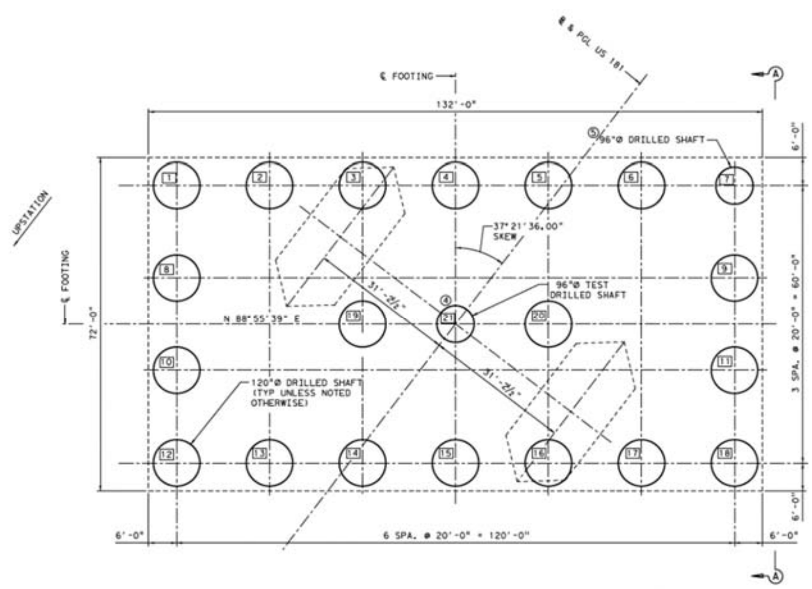
1. Texas Department of Transportation (TxDOT), "Technical Provisions for US 181 Harbor Bridge Project: Comprehensive Development Agreement." ["TP"]
2. American Association of State Highway and Transportation Officials (AASHTO), "LRFD Bridge Design Specifications," 7th Edition, 2014 with 2015 Interim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

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

3. Background

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

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



PLAN VIEW - TOWER 1NT FOUNDATION

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

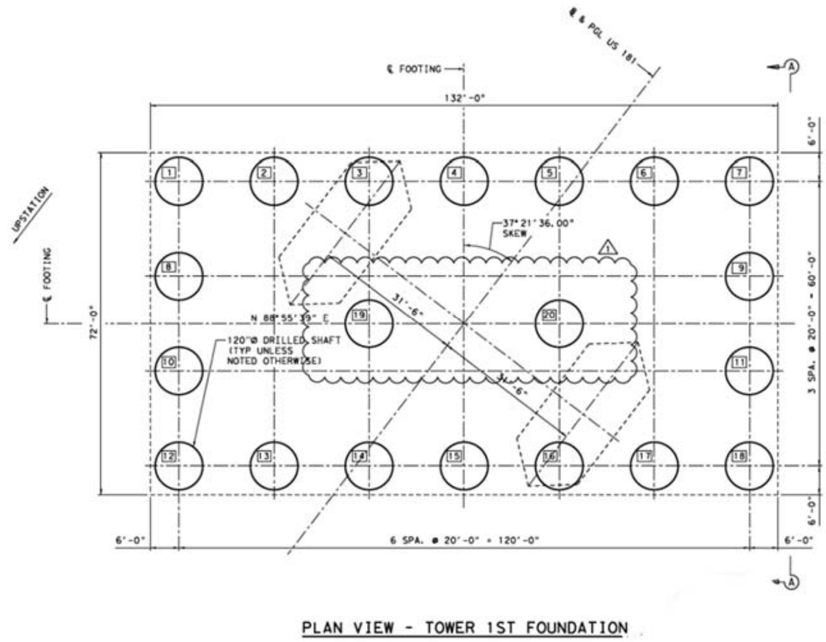


Figure 2: Foundation Cap 1ST, Foundation Cap and Shaft Layout, Drawing NHB-30B (by Others)

For all the analyses performed by the ISA, the participation of the 24"x24" driven piles at 1NT have been neglected, per the results and findings of an independent soil structure interaction analysis performed by the ISA's Geotechnical Engineer (see Appendix B of Reference 6. above).

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

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

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

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

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

As required by the Technical Provisions, TP §13.2.1.4, "The New Harbor Bridge shall be designed with an operational importance factor of 1.05. The operational importance factor shall be applied to the superstructure, including stay cables, and the towers." The tower foundations are intended to have the 1.05 factor applied, as it is a critical part of the towers' structural system. However, in the July 2022 Meeting, the DLE argued that the 1.05 importance factor does not apply to the tower foundations. The ISA team has concluded that it is not the intent of the Technical Provisions nor sound engineering to support an essential tower with a typical foundation. Nevertheless, this technical memorandum presents calculations both with and without the 1.05 importance factor. Although the Technical Provisions intended the tower foundations to be considered important, neglecting the 1.05 importance factor does not change the conclusions discussed herein.

4. Summary of Findings

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

- Axial capacity of the drilled shafts per AASHTO LRFD §10.5.5.2.4 and §10.8.3.5:
    - Foundation 1NT Factored ISA Geotechnical Capacity, 10' ϕ Shaft = 13,200 kips
    - Foundation 1NT Factored DLE Geotechnical Capacity, 10' ϕ Shaft = 13,300 kips
    - Foundation 1ST Factored ISA Geotechnical Capacity, 10' ϕ Shaft = 15,100 kips
    - Foundation 1ST Factored DLE Geotechnical Capacity, 10' ϕ Shaft = 15,400 kips
- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.05 and 90-deg wind:
    - Foundation 1NT, ISA Maximum Shaft Load = 17,159 kips, **D/C = 1.30**
    - Foundation 1ST, ISA Maximum Shaft Load = 18,431 kips, **D/C = 1.22**
    - Foundation 1ST, DLE Maximum Shaft Load = 17,857 kips, **D/C = 1.18**
- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.00 and 90-deg wind:
    - Foundation 1NT, ISA Maximum Shaft Load = 16,667kips, **D/C = 1.26**
    - Foundation 1ST, ISA Maximum Shaft Load = 17,926 kips, **D/C = 1.19**
    - Foundation 1ST, DLE Maximum Shaft Load = 17,418 kips, **D/C = 1.15**
- Axial Strength load demands for drilled shaft pile group with 1.05 Importance Factor – Individual Load Case and 90-deg wind:
    - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **8**
    - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **7**
    - Foundation 1ST, DLE Loads, # of Drilled Shafts with D/C > 1.00 = **5**

- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.05 and 270-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 17,484 kips, **D/C = 1.32**
  - Foundation 1ST, ISA Maximum Shaft Load = 17,951 kips, **D/C = 1.19**
  - DLE loads not provided for 270-deg wind
- Axial Strength load demands on the drilled shafts, per AASHTO defined load combinations with an Importance Factor of 1.00 and 270-deg wind:
  - Foundation 1NT, ISA Maximum Shaft Load = 16,960 kips, **D/C = 1.28**
  - Foundation 1ST, ISA Maximum Shaft Load = 17,502 kips, **D/C = 1.16**
  - DLE loads not provided for 270-deg wind
- Axial strength load demands for drilled shaft pile group with 1.05 Importance Factor – Individual Load Case and 270-deg wind:
  - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **10**
  - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **6**
- Axial strength load demands for drilled shaft pile group with 1.05 Importance Factor – Enveloped Load Cases (include wind loads at 90-deg and 270-deg):
  - Foundation 1NT, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **17**
  - Foundation 1ST, ISA Loads, # of Drilled Shafts with D/C > 1.00 = **13**

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

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

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

5. Conclusion

DRILLED SHAFT CAPACITY

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

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

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

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

RIGID CAP/PLASTIC SHAFT ANALYTICAL APPROACH

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

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

SUMMARY OF CONCLUSIONS

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

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

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

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

6. Response to DLE Tower Foundation Presentation

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

Comments in response to Document 277609-NHB-PRESE-Tower Foundations-00, 11-Aug-2022:

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

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

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

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

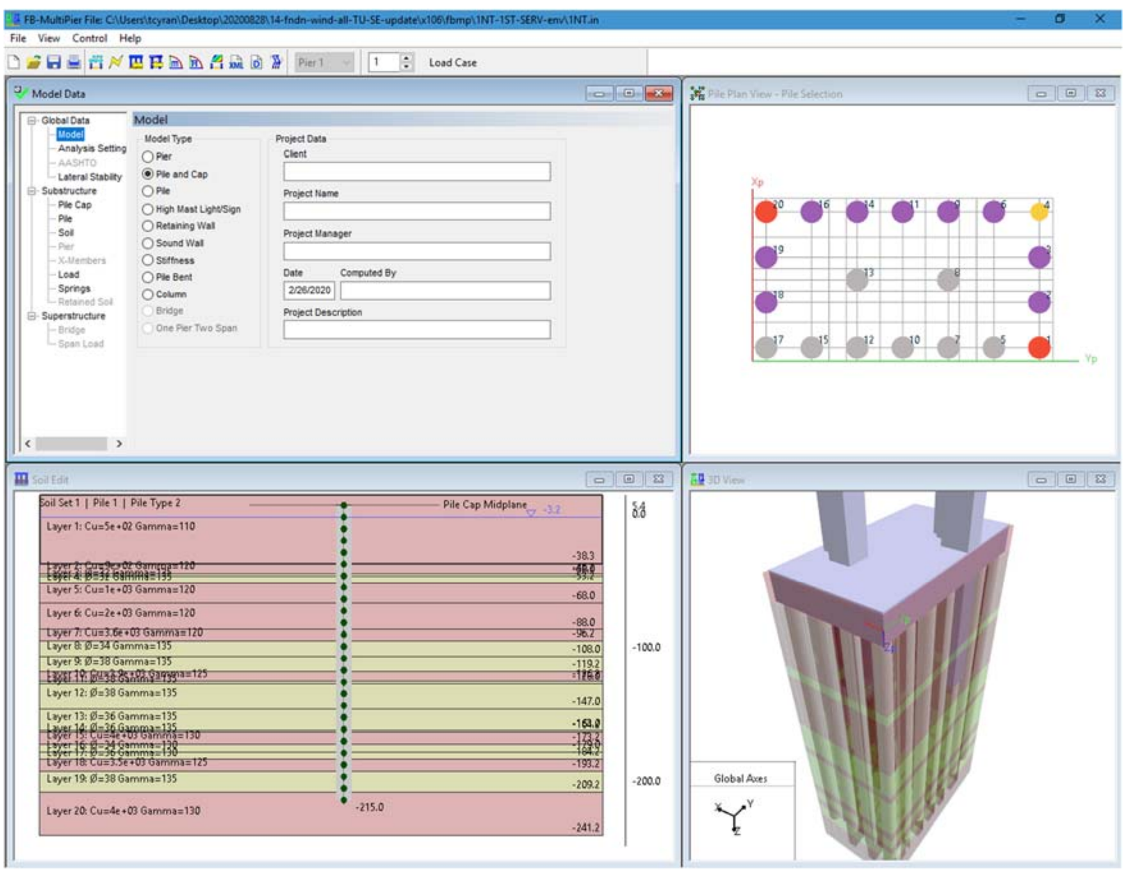
A.1 General Analysis Model Overview

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

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

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

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



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

A.2 Input Loads

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

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

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

90-Deg Wind Load Combinations

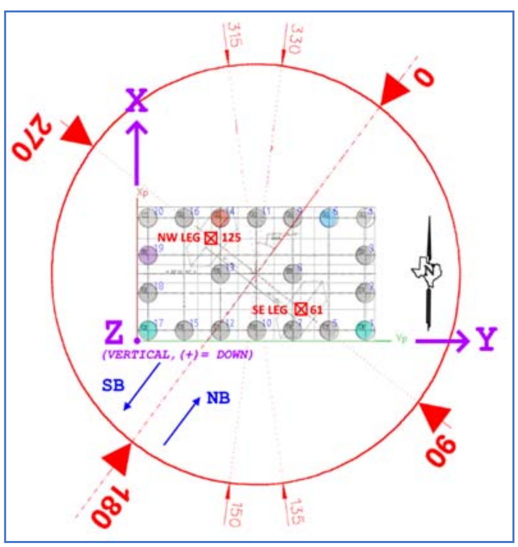
STRENGTH 3 LOAD COMBINATIONS -- IN DIRECTION OF FOUNDATION (SEE FIGURE BELOW)

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

270-Deg Wind Load Combinations

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

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



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

IBT TO CONFIRM THESE ARE CONCOMITANT

STRENGTH 3 LOAD COMBINATION

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

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

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

A.3 Drilled Shaft Loads

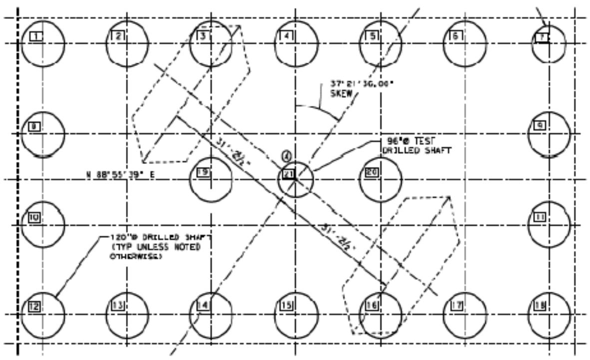
As noted in the body of this technical memorandum, the DLE has argued that the 1.05 importance factor should not apply to the tower foundations. The foundations are a critical part of the tower structural system, and so the 1.05 importance factor should be applied. However, neglecting the importance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

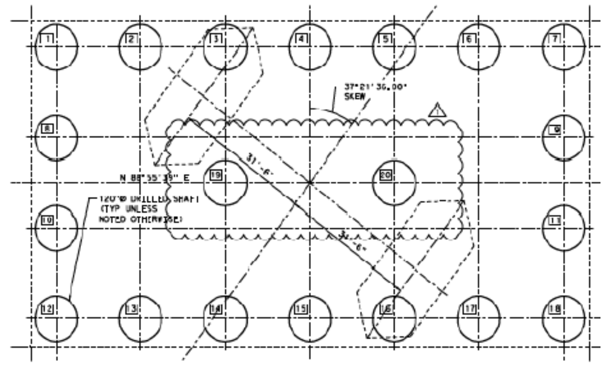
- ISA STR 3 Load Combination, 90-Deg Wind, Foundation 1NT/1ST, Factor 1.05
- ISA STR 3 Load Combination, 90-Deg Wind, Foundation 1NT/1ST, Factor 1.00
- DLE STR 3 Load Combination, 90-Deg Wind, Foundation 1ST, Factor 1.05
- DLE STR 3 Load Combination, 90-Deg Wind, Foundation 1ST, Factor 1.00
- ISA STR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

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

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

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





1NT

`90-degree, wind mode 24, IF = 1.05`

1ST

`90-degree, wind mode 23, IF = 1.05`

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

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

1NT

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

1ST

CASE 0: D/C RATIO

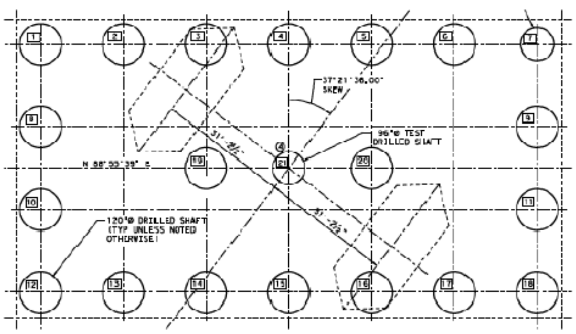
1.27	1.29	1.30	1.27	1.19	1.09	0.91
1.15						0.76
		1.09		1.00		
0.92						0.46
0.31	0.47	0.59	0.64	0.51	0.20	0.01

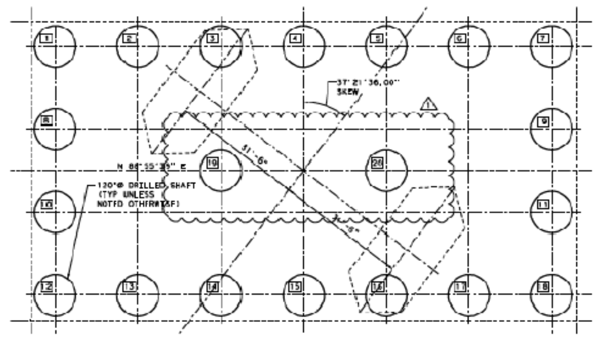
1NT

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

1ST

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





1NT

90-degree, wind mode 24, IF = 1

1ST

90-degree, wind mode 23, IF = 1

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

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

1NT

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

1ST

CASE 0: D/C RATIO

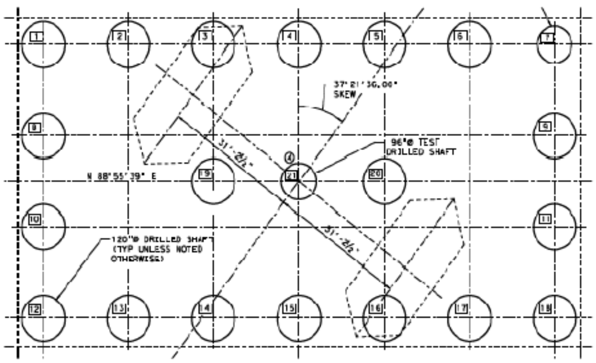
1.23	1.25	1.26	1.23	1.15	1.03	0.85
1.10						0.71
		1.05		0.96		
0.87						0.44
0.28	0.44	0.56	0.62	0.51	0.23	0.01

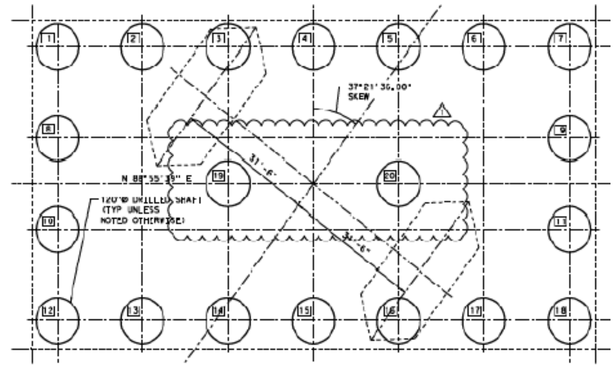
1NT

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

1ST

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





1NT

90-degree, IF = 1.05

1ST

90-degree, IF = 1.05

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

1NT

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

1ST

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

CASE 0: D/C RATIO

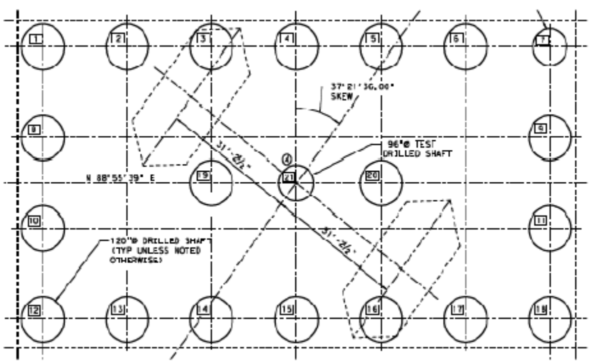
1NT

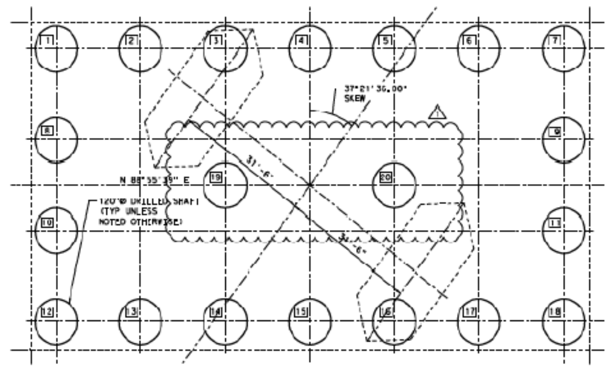
N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

1ST

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

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





1NT

90-degree, IF = 1

1ST

90-degree, IF = 1

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

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

1NT

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

1ST

CASE 0: D/C RATIO

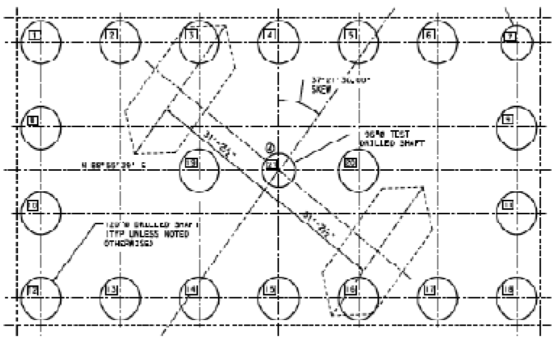
N/A	N/A	N/A	N/A	N/A	N/A	N/A
N/A						N/A
		N/A		N/A		
N/A						N/A
N/A	N/A	N/A	N/A	N/A	N/A	N/A

1NT

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

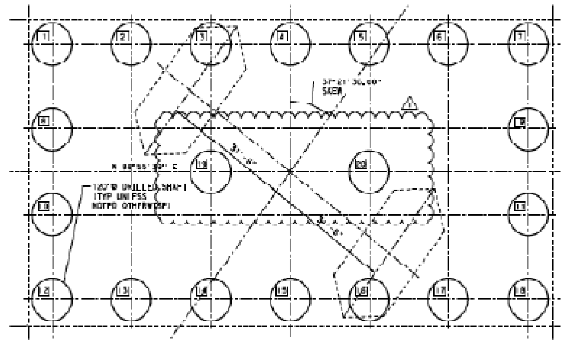
1ST

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



1NT

270-degree, wind mode 2, IF = 1.05



1ST

270-degree, wind mode 9, IF = 1.05

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

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

1NT

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

1ST

CASE 0: D/C RATIO

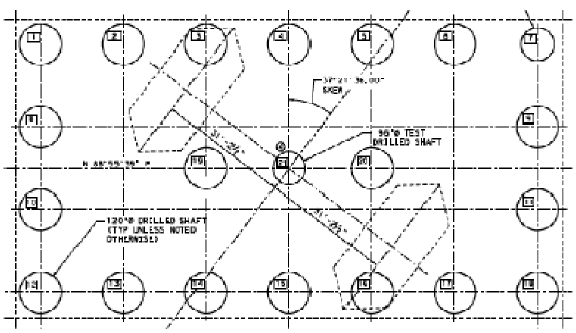
0.01	0.20	0.39	0.46	0.40	0.28	0.09
0.62						0.89
		1.02		1.09		
0.88						1.16
1.01	1.14	1.23	1.30	1.32	1.32	1.29

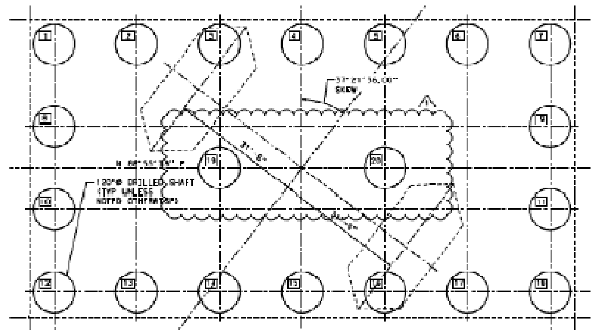
1NT

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

1ST

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





1NT — 270-degree, wind mode 2, IF = 1

1ST — 270-degree, wind mode 9, IF = 1

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

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

1NT

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

1ST

CASE 0: D/C RATIO

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

1NT

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

1ST

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

B.1 General Overview

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

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

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

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

B.2 ISA Technical Assessment

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

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

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

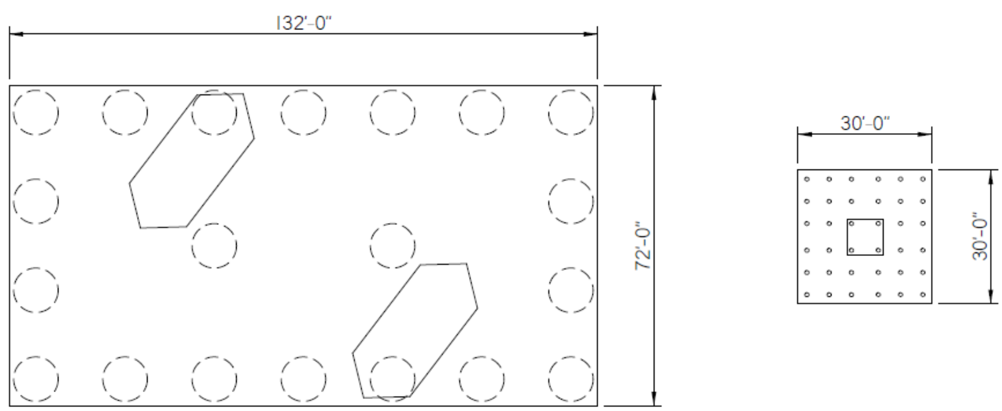
assumptions to envelope potentially uncertain behavior is acceptable; however, every element of the structure must be shown to have adequate capacity for the results of each model.

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

Duan and McBride Analysis

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

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



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

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

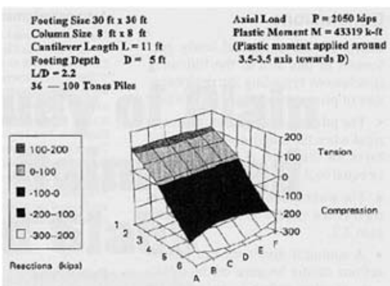


Fig. 6 — Pile reactions of pile cap with depth $D = 5$ ft.

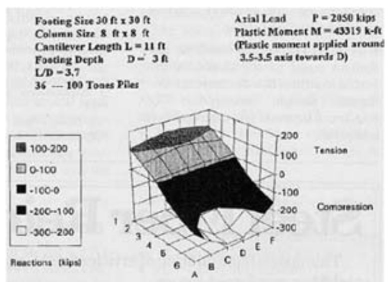


Fig. 7 — Pile reactions of pile cap with depth $D = 3$ ft.

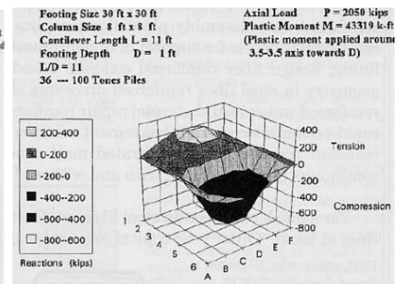
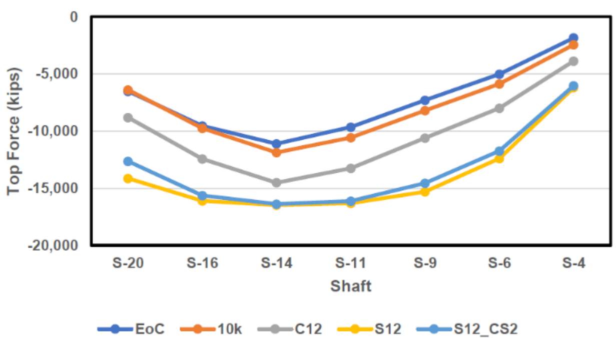
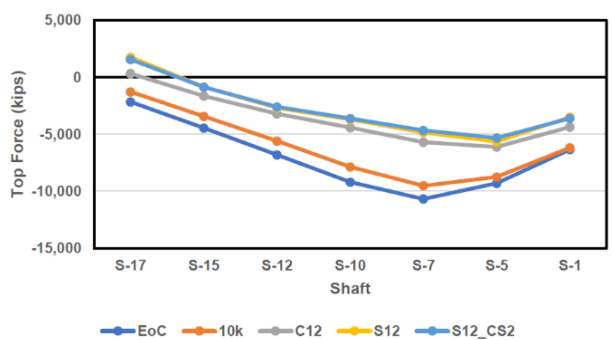


Fig. 8 — Pile reactions of pile cap with depth $D = 1$ ft.

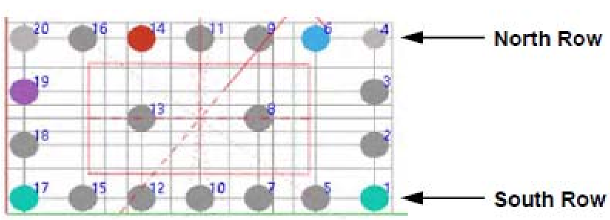
Using the results from the ISA Phase 1 report, similar shaft load plots are generated from a detailed FLAC model of the foundation with soil non-linearity that accurately captures the interaction between soil, shafts, and foundation cap. These plots are shown below for multiple load stages at 1NT.



North Row



South Row



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

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

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

B.3 Conclusions

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

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

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

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

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