Keith Armstrong
Project Manager

August 30, 2022

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Texas Department of Transportation
Office of General Counsel
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RE: US 181 Harbor Bridge Replacement Project
CSJ No.: 0101-06-095
FDLLC Response to TxDOT Notice of Developer Default – Proposed Schedule of Design Modifications and Action Plan

Dear Mr. Williams,

As a follow up to our letter dated August 28, 2022 in response to TxDOT's Notice of Developer Default from August 16, 2022, FDLLC has prepared a summary schedule and an action plan to implement the design modifications as part of the commitments offered by FDLLC to address TxDOT's concerns with the design of the New Harbor Bridge.

Please note that this schedule and action plan are preliminary and a work in progress subject to TxDOT's acceptance of FDLLC's offer.

Respectfully,

Keith Armstrong
Project Manager

cc: Valente Olivarez, P.E. – TxDOT (e-mail only)
Joseph Briones, P.E. – TxDOT (e-mail only)
José Antonio López-Monís Plaza – Dragados USA, Inc. (e-mail only)
Javier Sevilla – Flatiron Constructors, Inc. (e-mail only)
Kurt Knebel – Flatiron Constructors, Inc. (e-mail only)
Justo Molina – FDLLC (e-mail only)
Jaime Hurtado Cola – Dragados USA, Inc. (e-mail only)
Kyle Bogdan – Flatiron Constructors, Inc. (e-mail only)
Aconex

Enclosures:

1. Proposed Schedule of Design Modifications
2. Proposed Action Plan

PROJECT: NEW HARBOR BRIDGE

Design Modification Schedule

2022: JUL AUG SEP OCT NOV DEC | 2023: JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC | 2024: JAN FEB MAR APR MAY JUN JUL AUG SEP OCT NOV DEC | 2025: JAN FEB MAR APR MAY JUN

"Design Modification" #1: Tower Drilled Shafts & Cap			
Design & Approval	15-Sep-22	15-Jan-23	10 weeks to complete design + 2 weeks TxDOT review + 2 weeks to address comments + 2 weeks final approval = 16 weeks.
North Pylon: "Design Modification" work			
- Port Agreement on the Scope of Work	15-Sep-22	26-Oct-22	
- Dock demolition: plan & removal	29-Sep-22	22-Feb-23	
- Drilled Shafts	26-Feb-23	5-Apr-23	
- Cofferdam & excavation	6-Apr-23	2-Jun-23	
- Footing	5-Jun-23	25-Aug-23	
North Pylon			
- Remobilization	15-Sep-22	15-Dec-22	
- Pylon: Upper Pylon Lifts #6 to #10	16-Dec-23	4-Apr-23	
- Deck: Segment erection #5 to #31	16-Dec-23	14-Oct-23	*Segment #31 erection after completion of "Design Modification" work.
- Deck: Segment erection #31 to #86	15-Oct-23	2-Aug-24	
- Deck: Finishes	3-Aug-24	12-Jan-25	
South Pylon: "Design Modification" work			
- Port Agreement on the Scope of Work	15-Sep-22	26-Oct-22	
- Dock demolition: plan & removal	29-Sep-22	22-Feb-23	
- Drilled Shafts	26-Feb-23	5-Apr-23	
- Cofferdam & excavation	6-Apr-23	2-Jun-23	
- Footing	5-Jun-23	25-Aug-23	
South Pylon			
- Remobilization	15-Sep-22	15-Dec-22	
- Pylon: Upper Pylon Lifts #4 to #10	16-Dec-23	21-May-23	
- Deck: Segment erection #2 to #31	16-Dec-23	5-Nov-23	*Segment #31 erection after completion of "Design Modification" work.
- Deck: Segment erection #31 to #86	6-Nov-23	18-Aug-24	
- Deck: Finishes	19-Aug-24	12-Jan-25	
"Design Modification" #2: Delta Frames			
Design & Approval	15-Sep-22	15-Nov-22	2 weeks to complete design + 2 weeks TxDOT review + 2 weeks to address comments + 2 weeks final approval = 8 weeks.
"Design Modification" work	16-Nov-22	15-Feb-23	
Cast segments and delta frames	15-Dec-22	19-Oct-23	
"Design Modification" #3: Bearing Uplift			
Design & Approval	15-Sep-22	28-Feb-23	4 months to complete design + 2 weeks TxDOT review + 2 weeks to address comments + 2 weeks final approval = 5.5 months.
"Design Modification" work	1-Mar-23	30-Jun-23	
"Design Modification" #4: Erection Loading			
Design & Approval			Design schedule not defined - not needed to resume works at the pylons.
"Design Modification" work			10/14/2023 - Install strain gauge on segment #31.

15-July-22 NHB Works suspended

25-April-24 PBS-5 Baseline Phase 1 completion

12-Jan-25 New Phase 1 completion

8.5 months

Flatiron Dragados LLC

US181 Harbor Bridge Replacement Project

Proposed Action Plan

277609-NHB-REP-Action Plan

02 | August 30, 2022

Job number 277609

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Appendices

1 Introduction

The Texas Department of Transportation ("TxDOT") has entered into a Comprehensive Development Agreement ("CDA") with Flatiron / Dragados LLC ("FDLLC") ("Developer") ("Contractor") to develop, design , construct and maintain the US 181 Harbor Bridge Replacement Project, which extends north-south along US 181 and SH 286 and east-west along I-37, and includes: US 181 at Beach Avenue on the north; SH 286 at Morgan Avenue on the south; I-37 and Up River Road on the west; and I-37 and Shoreline Boulevard on the east (the "Project"); including the maintenance of the Project for 25 years.

The New Harbor Bridge is defined in the CDA and is the cable supported bridge spans over the Corpus Christi Ship Channel that support US 181, including all associated Elements such as towers, substructures, and foundations supporting the main span and back spans.

The Project also includes Approaches and Roadworks.

The Arup-CFC Design Joint Venture ("Arup-CFC") ("Designer") was approved by TxDOT as the new Lead Engineering Firm, responsible:

- to complete that portion of the Design Work assigned to Figg ("Previous Designer") with regard to the New Harbor Bridge, and
- to ensure that all engineering and Design Work performed by Previous Designer with regard to the New Harbor Bridge is reviewed and signed/sealed by the replacement Lead New Harbor Bridge Engineer ("Engineer of Record")

Arup-CFC has executed a design Services Agreement with FDLLC to discharge those responsibilities in addition to other professional services associated with the Project.

TxDOT issued a Notice of Nonconforming Work on April 29, 2022, followed by a direction to suspend work on the erection of the main span superstructure on July 15, 2022, and a Notice of Developer Default on August 16, 2022. FDLLC has requested Arup-CFC to provide an action plan of design modifications that act upon IBT's conclusions with a view to doing what is necessary to satisfy TxDOT.

2 Proposed Modifications

The proposed design modifications and actions are summarized below:

- Extend the footings adjacent to the tower legs and add additional drilled shafts to each tower.
- Add longitudinal and transverse reinforcement to the top of the in-situ concrete joint between the delta frame and adjacent precast segment and make continuous into both precast units
- Modify the bridge design to prevent bearing decompression from occurring at strength limit state
- Establish limiting values of tension strain in the bottom flange of the superstructure above the temporary pier and monitor during construction
- Recommence meetings and dialogue to resolve any other items of concern

We note that Port of Corpus Christi Authority ("PCCA") approvals will be required for the modifications of the foundation elements that support the towers.

3 Tower Drilled Shafts

The proposed plan for the tower drilled shafts is:

- **Action Regarding Notice of Nonconforming Work Item 2 |** Extend the tower footings adjacent to the tower legs and add additional drilled shafts to each tower.

Further description of the proposed design modification is provided below.

3.1 Additional drilled shafts

Appendix A of this report includes a preliminary drawing of this proposed design modification for the North Tower. The South Tower will be similar but with a reduced extension and reduced number of piles.

4ft diameter has been selected for the additional drilled shafts due to the reduced mobilization time associated with a rig of this size. With reference to AASHTO LRFD C10.8.3.5.6, the shaft diameter remains within the range of applicability of the existing load tests. Based on these tests, the calculated factored capacity of each drilled shaft is 4,400 kips.

Figure A5 1NT of IBT's TM 1001 was used as a basis for determining an additional drilled shaft arrangement that it is considered will be acceptable to TxDOT based upon the design criteria and demands presented by IBT. For the North Tower, a total of seven additional drilled shafts are proposed in a line along each long edge of the footing in the vicinity of the tower leg.

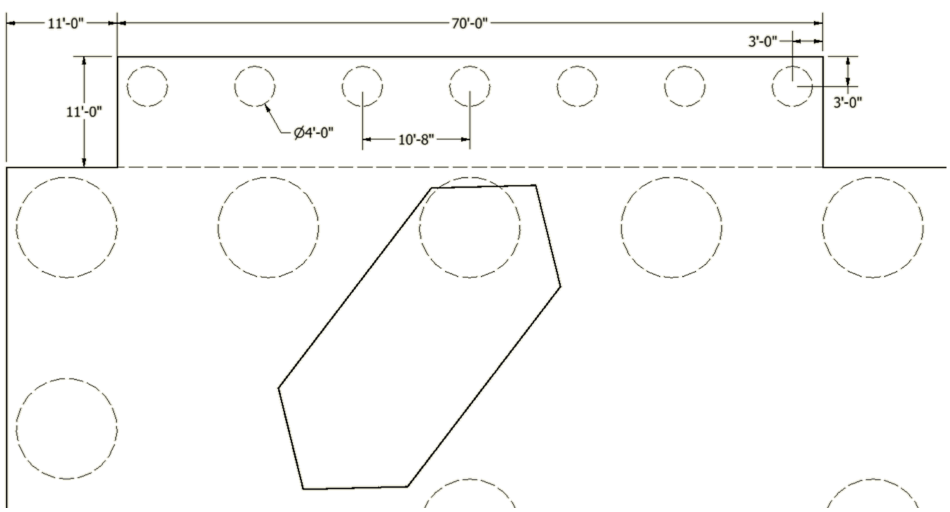


Figure 1 | Layout of additional drilled shafts

An extension to the footing will be provided. This will be attached by bars which are drilled and fixed into the existing footing.

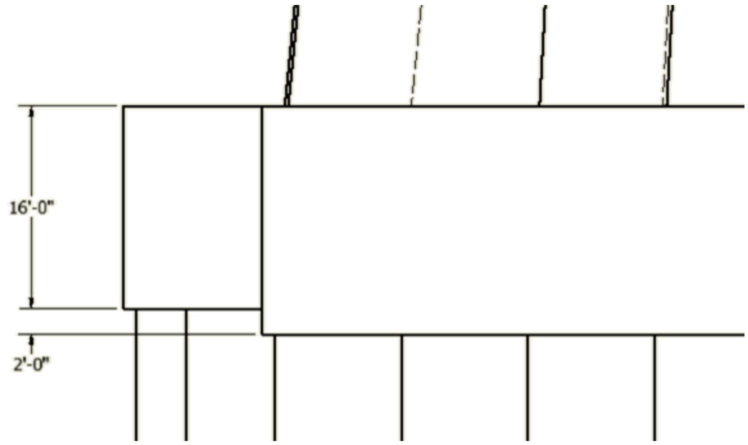


Figure 2: Section through footing extension

The placement of these bars is critical to avoid damage to the existing reinforcement. The following measures may be taken:

- The interface will be prepared to the specified roughness and the side bars will be located and mapped.
- The existing shear reinforcement (on a 2ft x 2ft grid) will be mapped and all drill and fix bars will be centrally located in the gaps between these bars.
- The main tension steel (bottom) will be located above the bottom mat of the existing footing and will only be installed in the gaps between the drilled shafts so that there are no conflicts with the projecting shaft reinforcement
- The shear friction reinforcement will be curtailed as necessary to avoid conflicts with the tower leg dowel bars.

As shown in Figure 3, the tower foundations were constructed within cofferdams and adjacent to the Port of Corpus Christi bulkheads. The cofferdam and bulkheads will require modification which is a relatively complex construction operation and will require coordination with, and approvals from, the Port of Corpus Christi Authority.

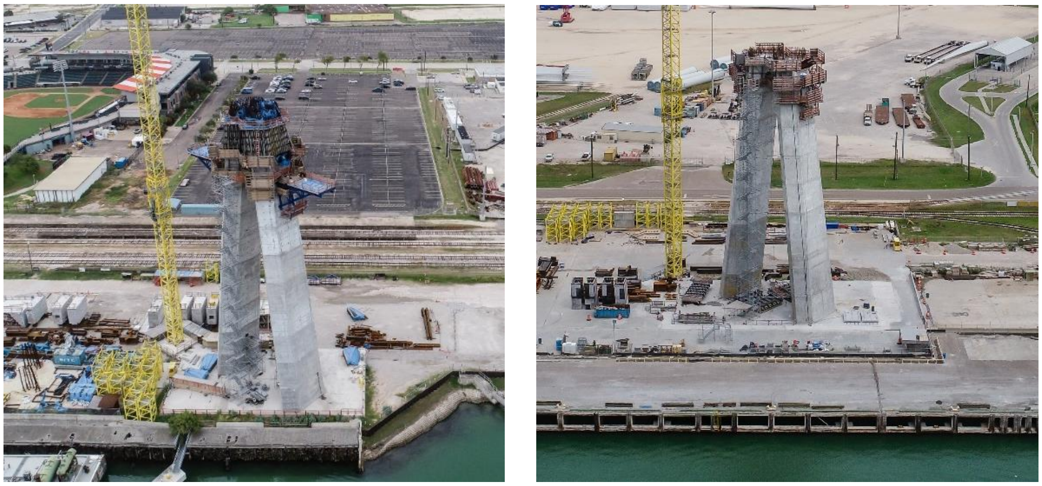


Figure 3: Proximity of footing to bulkheads (left: south tower, right: north tower).

A preliminary Plaxis analysis of this modified foundation has been carried out for tower NT1 with input loads taken as the 270 degree wind load combination tower leg loads provided by IBT with an operational importance factor of 1.05. This produces the largest reported demand / capacity ratio in IBT TM 1001. The analysis models the flexibility of the footing and non-linear response of the ground.

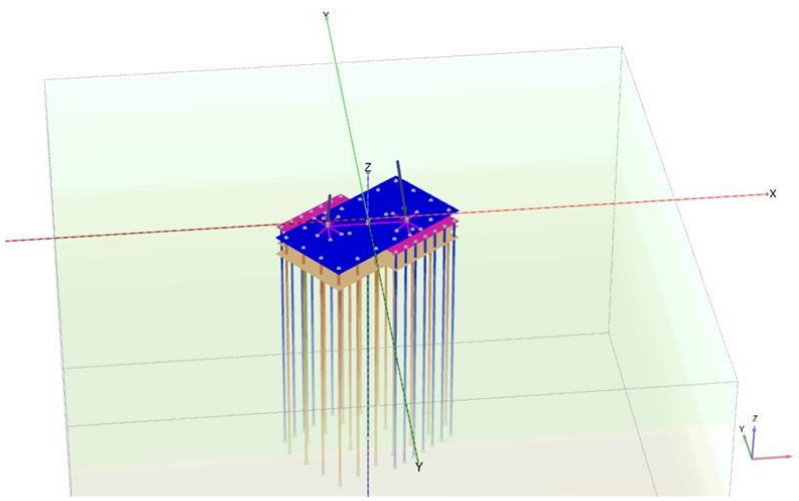


Figure 4 | Plaxis model of modified foundation (NT1)

The analysis includes consideration of staged construction. Loads representing the current state of construction are first applied to the current foundation and then the additional foundation elements are activated. Finally, the additional loads to reach the loads in IBT TM 1001 are applied.

The deformed shape of the footing is shown in Figure 5. Drilled shaft loads and displacements are shown in Figure 6 and Figure 7 respectively. All shaft loads are less than the factored capacity and all shaft displacements are less than 5% of shaft diameter.

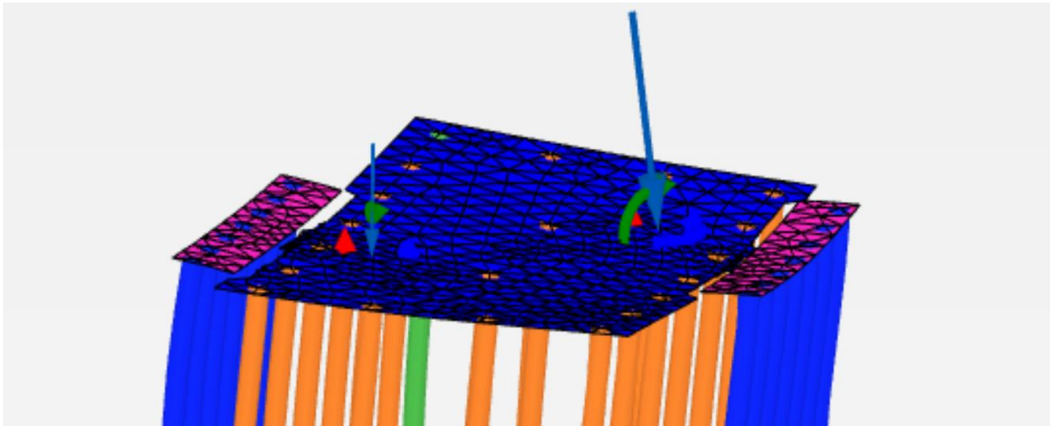


Figure 5: Deformed shape of the footing (1NT 270 deg. IBT input loads, OIF = 1.05)

Additional drilled shafts

Demand	-101	-31	1	14	39	34	5	kips
D/C Ratio	Tension	Tension	0%	0%	1%	1%	0%	

5483 kips 41%	5828 kips 44%	6003 kips 45%	6036 kips 45%	5480 kips 41%	4995 kips 38%	3694 kips 28%
6929 kips 52%						7640 kips 57%
		8082 kips 61%		8446 kips 64%		
7849 kips 59%						10283 kips 77%
10263 kips 77%	10638 kips 80%	11232 kips 84%	12210 kips 92%	12646 kips 95%	12335 kips 93%	12588 kips 95%

3418	3491	3668	3728	3721	3732	3914	kips
78%	79%	83%	85%	85%	85%	89%	

Figure 6: Drilled shaft loads (1NT 270 deg. IBT input loads, OIF = 1.05)

Additional drilled shafts

Displacement	-0.7	-0.7	-0.8	-0.8	-0.8	-0.8	-0.8	in
% Pile Diameter	1.4%	1.5%	1.6%	1.7%	1.7%	1.8%	1.7%	

-0.8 in 0.7%	-0.9 in 0.8%	-1.0 in 0.8%	-1.0 in 0.9%	-1.0 in 0.9%	-1.0 in 0.8%	-0.9 in 0.7%
-1.0 in 0.9%						-1.2 in 1.0%
		-1.4 in 1.2%		-1.5 in 1.2%		
-1.2 in 1.0%						-1.4 in 1.2%
-1.3 in 1.1%	-1.5 in 1.3%	-1.7 in 1.4%	-1.8 in 1.5%	-1.9 in 1.6%	-1.8 in 1.5%	-1.7 in 1.4%

-1.9	-2.0	-2.1	-2.1	-2.0	-2.0	-1.9	kips
4.0%	4.1%	4.3%	4.3%	4.2%	4.1%	3.9%	

Figure 7: Drilled shaft displacements (1NT 270 deg. IBT input loads, OIF = 1.05)

3.2 Schedule

Design development of the foundation modification should take place with weekly over the shoulder meetings with IBT to ensure that the detailing is being developed in a manner that will address IBT's concerns. We estimate that it would take approximately four weeks to agree on a finalized design concept and a further six weeks to prepare final modified drawings.

We propose that erection of the superstructure should be permitted to continue in parallel with construction of the modifications to the tower foundations up to an agreed point which should be at least as far as closure with the temporary pier.

A more detailed discussion will take place to allow all parties to agree on this plan. However, shown below is a comparison of the tower leg loads for different conditions which illustrates why this course of action is a reasonable way to mitigate schedule delay.

Construction Stage	**Factored Tower Leg Load**	**Notes**
Current condition C1_Seg 02-05 PT	18,300 kips (without wind)	1.25 x EM3 stage by stage load
Just before temporary pier closure C6_Seg 01 PT	41,000 kips (without wind)	1.25 x EM3 stage by stage load
	47,800 kips (with wind)	STRIII critical stage
Just after temporary pier closure C6_TS & TN closure	40,800 kips (without wind)	1.25 x EM3 stage by stage load
In-Service	114,000 kips (with wind)	STR III (IBT loads)

The axial load in an individual tower leg is the most significant influence of demands on the foundation. If superstructure erection were to continue as far as closure with the temporary pier the most critical stage would be just prior to closure where wind forces would be relatively large due to the unrestrained cantilever (closure with the temporary pier provides restraint). This demand is approximately 40% of the governing in-service load.

According to the PBS5 Rev 03 Schedule Update 80 (which was the last update before the suspension of work) the first temporary pier closure was scheduled to take place in February 2024, representing at least seven months of construction work that could reasonably be carried out in parallel with the foundation modifications.

The Plaxis analysis described above was re-run based on a staged analysis assuming the above (i.e. a larger load is applied to the current foundation before the additional foundation elements are activated). Drilled shaft loads and displacements are shown in Figure 6 and Figure 7 respectively. There is a slight redistribution of load with both the existing 10ft diameter shafts and the additional 4ft diameter shafts remaining within the strength limit state capacity and displacement limits. The tension in the additional shafts on the side away from the maximum load is well within their capacity.

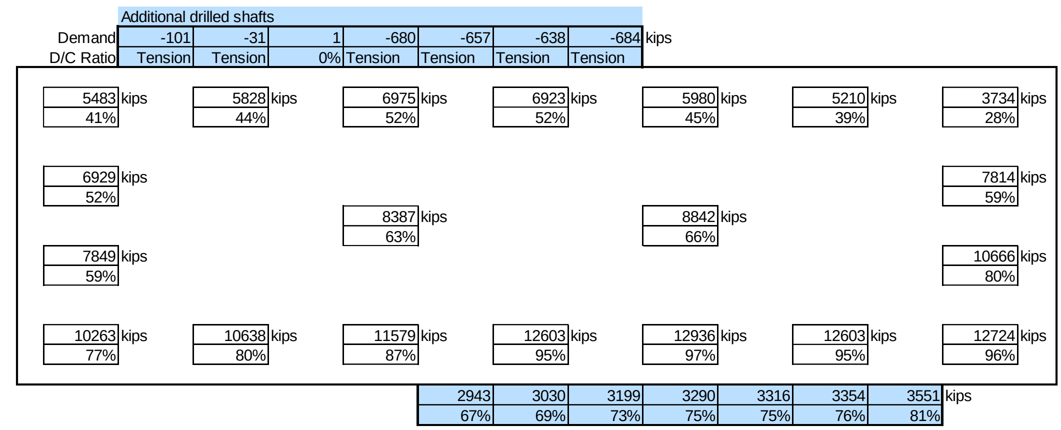


Figure 8: Drilled shaft loads (1NT 270 deg. IBT input loads, OIF = 1.05, additional foundation elements activated at a later stage)

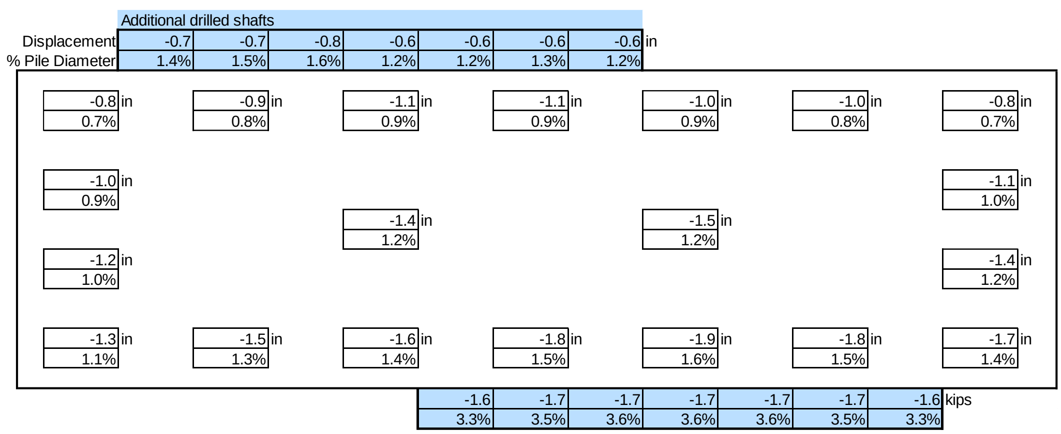


Figure 9: Drilled shaft displacements (1NT 270 deg. IBT input loads, OIF = 1.05, additional foundation elements activated at a later stage)

4 Tower Foundation Cap

The additional drilled shafts and footing extension which have been proposed to address the first primary item achieve this strength increase and no further design modifications are required. This is briefly described in the following section.

4.1 Influence of proposed modification to tower foundation on two-way shear action

Figure 10 shows three perimeters for two way shear action for the modified foundation. Note that these perimeters were determined by extending the critical perimeter shown by IBT in Appendix A3 of TM 1002.

The tower footing extension will be 5.5 ksi concrete and will be provided with six legs of #11 shear reinforcement at 12" centers which will be sufficient to achieve the maximum resistance allowed by AASHTO LRFD Eq. 5.13.3.6.3-2. The factored shear resistance of the footing extension will be approximately 8,700 kips and 2.75 additional drilled shafts are contained within Perimeter 1.

Based on IBT's application of AASHTO LRFD 5.13.3.6.1, Perimeter 1 adds the following to the capacity:

$$2 \times 8{,}700 + 2.75 \times 4{,}400 = 29{,}500 \text{ kips}$$

Perimeter 2 is less critical and adds approximately 30,300 kips to the capacity.

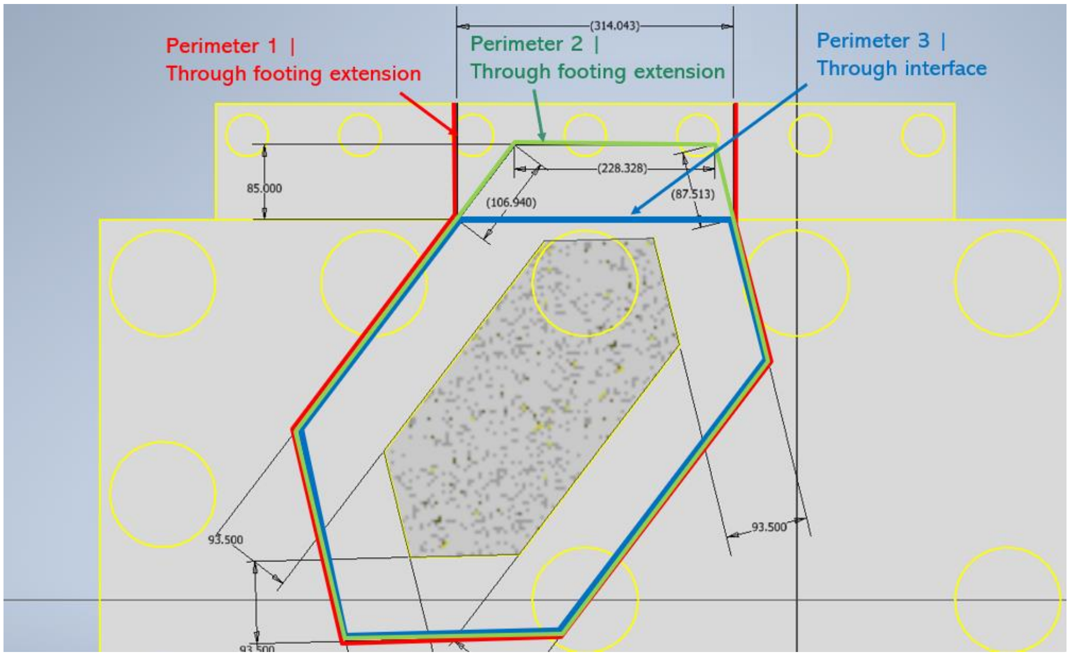


Figure 10 | Critical perimeters for two way action based on IBT Technical Memorandum

The interface between the footing and the footing extension will be designed with a factored interface shear capacity of at least 28,000 kips over the region shown in Figure 10 so that Perimeter 3 adds at least 28,000 kips to the capacity. As such:

- The footing extension provides at least an additional 28,000 kips of capacity which is more than the shortfall calculated by IBT in Appendix B3 of TM 1002.
- The 28,000 kips of additional capacity is approximately 25% of the maximum leg load. Therefore, the leg is supported on all four sides and IBTs concerns over the proximity of the leg to the edge of the cap are addressed.

This strengthening of the footing will also address IBT's concerns regarding one-way action and bending over a half-section of the footing.

4.2 Schedule

Refer to Section 3.2.

5 Delta Frames

The proposed plan for the delta frames has two components:

- **Action Regarding Notice of Nonconforming Work Item 10 |** Provide longitudinal and transverse reinforcement in the top of the in-situ concrete joint between the delta frame and adjacent precast segment and make continuous into both precast units
- **Action Regarding Notice of Nonconforming Work Items 11 to 14 |** Continue to engage with TxDOT / IBT to determine whether any other areas of concern related to the delta frames remain

5.1 Additional reinforcement

Details of a proposed design modification which introduces additional reinforcement are provided below.

We propose to provide (8) eight #4 bars in the transverse direction (across the joint) and (2) two #4 bars in the longitudinal (secondary) direction.

The transverse reinforcement will be cranked in the delta frame end to run parallel to the existing reinforcement and avoid clashes as shown in Figure 11. The existing bars will be located and mapped prior to drilling to avoid clashes.

The transverse reinforcement would be drilled and epoxied into the precast segment and delta frame on either side of the cast-in-place (CIP) joint. A minimum embedment length of 12" (tension development length for #4 bar to AASHTO LRFD 5.11.2) will be provided. The reinforcement will overlap and terminate within the CIP joint with a standard hook following the detailing requirements of AASHTO LRFD Figure C5.11.2.4-1.

Figure 12 shows how the proposed reinforcement arrangement would be installed to ensure ease of constructability. The bars which are continuous into the precast box girder segment would be drilled and fixed on the ground. The bars which are continuous into the delta frame would be epoxied into predrilled holes after the delta frame is in position. The secondary bars would be tied out of place and then moved into position after the delta frame is installed.

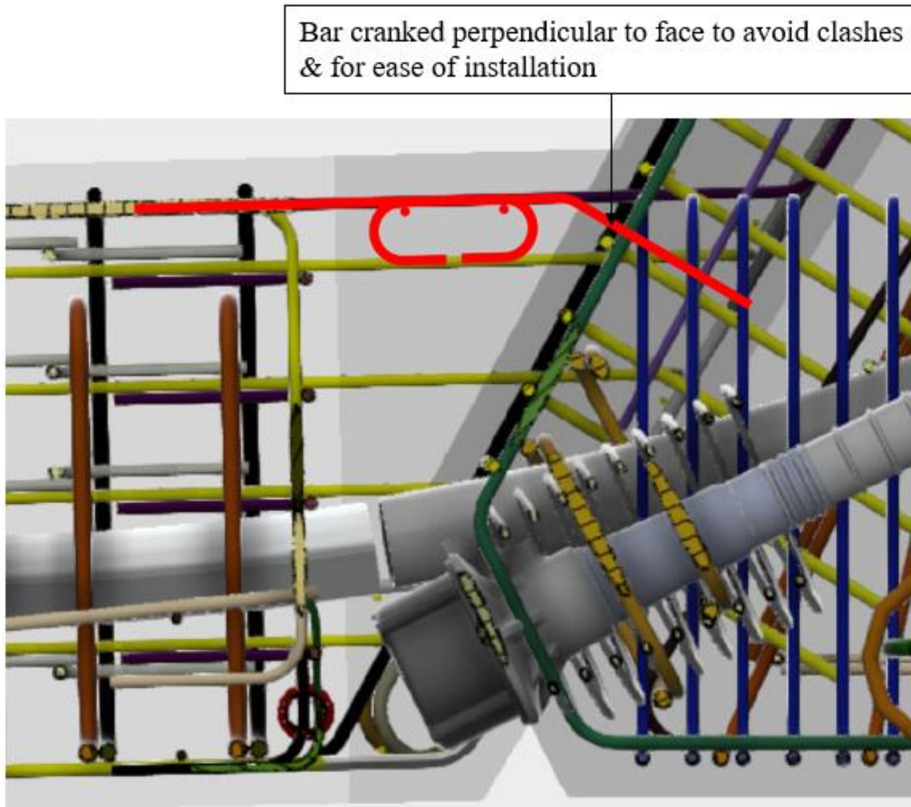


Figure 11. Proposed reinforcement layout. New reinforcement shown in red, all other reinforcement is existing.

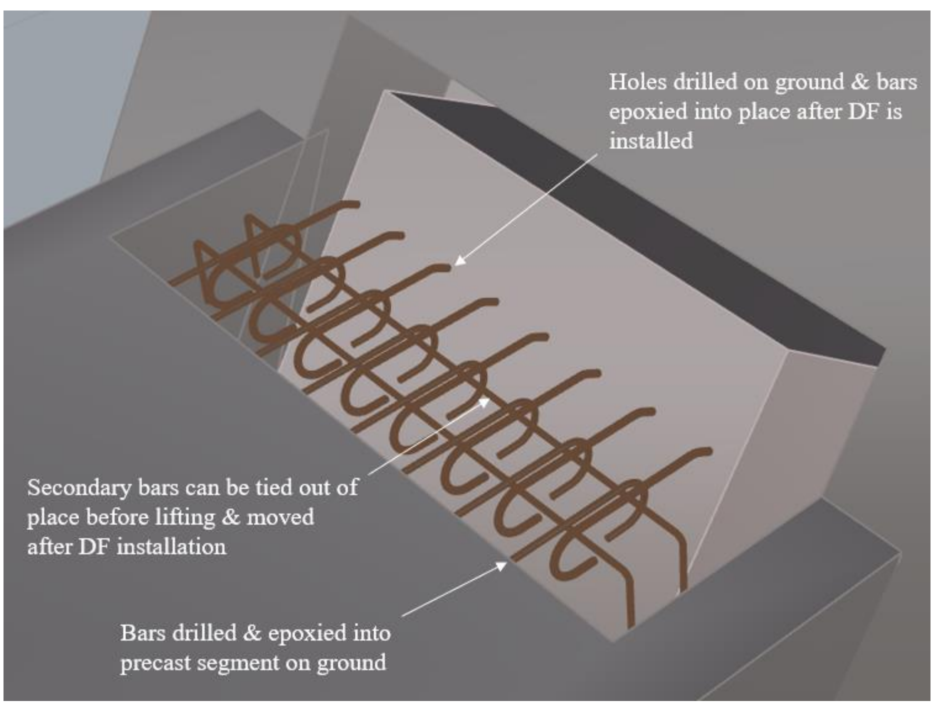


Figure 12. Proposed reinforcement installation sequence

The quantity of reinforcement has been calculated based on the tension block for the Service I load combination and with the stress in the reinforcement being limited to be less than 0.5 f_y. Since the transverse stresses at the top of the joint do not exceed 0.0948 $f'_c{}^{0.5}$ this is consistent with IBT's use of AASHTO LRFD 5.9.4.2.2 for segmentally constructed bridges if the location were "Longitudinal Stresses through Joints in the Precompressed Tensile Zone."

The characteristics of the cast-in-place pour-back material was specified on drawing NHB 0B Rev 4 which was issued as part of Notice of Design Change ("NDC") 0512 dated 9/17/2021. For convenience, the specification is reproduced in Figure 13.

5.0 MATERIALS (CONT.)
A. (CONT.)
7. IN-SITU STITCH BETWEEN DELTA FRAME & SEGMENTS TO BE FORMED WITH HIGH PERFORMANCE FIBER REINFORCED CONCRETE MEETING THE FOLLOWING MINIMUM REQUIREMENTS:
- MIN. COMPRESSIVE STRENGTH = 10 KSI AT 56 DAYS
- MIN. TENSILE STRENGTH = 0.6 KSI AT 56 DAYS
- NON-REACTIVE AGGREGATE, MAX. SIZE = 1/4"
- LONG-TERM SHRINKAGE < 400 MICROSTRAIN AT 56 DAYS
- MODULUS OF ELASTICITY > 6000 KSI
- SUITABLE FOR MIN. APPLICATION THICKNESS = 1" / MAX. APPLICATION THICKNESS = 3'-6"

Figure 13 | Extract from drawing NHB 0B Rev 4

5.1.1 Schedule

It is estimated that the time required for reinforcement procurement and installation on the first delta frame would be less than the time required to remobilize the 1,350-ton crawler crane that is used to erect the delta frame itself. The reinforcement can be progressively installed on delta frames in the order that they are needed for construction.

5.2 Continued engagement

Arup-CFC has provided presentation slides to TxDOT / IBT regarding Items 11 to 14 on July 29, 2022. We propose that a meeting should be held to review and discuss this material.

5.2.1 Schedule

We are available as soon as a meeting can be scheduled.

Once we understand whether any other areas of concern related to the delta frames remain we will be able to propose a plan and schedule to address those concerns (if any).

6 Bearing Uplift

The proposed plan for the bearing uplift is:

- **Action Regarding Notice of Nonconforming Work Item 16 |** Modify the bridge design such that bearing uplift does not occur at strength limit state.

6.1 Modify bridge design to prevent uplift at strength limit state

There are a number of alternatives available to avoid uplift at strength limit state. Based on investigations carried out to date the preferred design modifications are anticipated to be:

- Increase the size of the exterior hold down cables at the transition piers.
- Stiffen the bridge transversely (if required) by introducing a steel beam at the end closure pour.
- Modify the design of the transition pier diaphragm, bearings and superstructure transition segment accordingly.
- Verify the transition pier and pier cap for the revised load distribution.

Modifications will be required to the transition pier diaphragm which has already been built. This will require partial demolition, reinforcement adjustment and pour back of new concrete in a relatively confined space. We have reviewed the DSI shop drawings for the hold downs and the slot in the pier cap may also need to be enlarged.

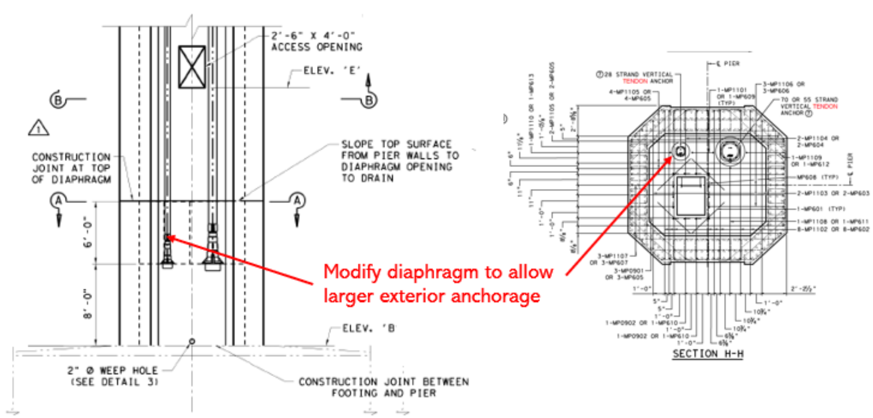


Figure 14 | Modifications will be required to the already built transition pier diaphragm

Bearing uplift at strength limit state occurs for the interior bearings under Strength III loading. This is because the stay cable system, which pulls upwards when the main span is subject to wind buffeting, is located along the center of the bridge.

The hold down system was designed with larger cables towards the center of the bridge to counteract the upwards pull along the center. This design was sufficient to prevent bearing uplift at service limit state. However, it is not sufficient to prevent bearing uplift at strength limit state.

The design modification proposes to have more balanced hold down sizes. However, the upwards pull during governing loading conditions will still be concentrated towards the middle of the main span. This will be counteracted by creating a transverse flexure of the bridge using strong backs prior to pouring the in-situ concrete stitch that closes the superstructure cantilever with the transition pier segment. When the strong backs are released after the stitch has cured a compression will be released into the interior bearings.

The relative force distribution between inner and outer bearings is controlled by the transverse stiffness of the bridge. A stiffer bridge pulls more equally on the four bearings. If required the transverse stiffness can be increased with a steel beam which would be connected to the superstructure by the in-situ closure pour.

6.1.1 Schedule

We estimate that it will take approximately four months to incorporate the above modifications.

Since the design modifications are localized to areas of the bridge remote from the main construction front superstructure erection can continue whilst this work is being carried out.

7 Erection Loading

7.1 Issue related to wind loads

The IBT Technical Memorandum TM 1005 refers to strength limit state loads on the back span cantilever immediately before closure with the backspan pier. Please note that a new version of the Erection Manual was issued on August 12, 2022.

The latest version of the Erection Manual (Revision 03) has, amongst other things, retuned the bridge to produce a negative dead load moment at the critical location above the temporary pier prior to closure. Based on the loads due to the erection sequence documented in the current version of the Erection Manual there is no exceedance of strength limit state demands at this location.

Arup-CFC proposes the following action:

- **Action Regarding This Item |** Establish limiting values of tension strain in the bottom flange of the superstructure above the temporary pier that maintain sufficient margin for a strength limit state wind event and install strain gauges and monitoring at this location to ensure that actual stresses during construction do not exceed predicted values in the Erection Manual.

7.1.1 Schedule

This action is not schedule critical and we therefore propose to prioritize other actions. We propose to issue an Erection Manual update that includes the limiting values of tension strain as well as the requirements for monitoring during construction within 12 weeks of TxDOT / IBT confirming that this action adequately addresses this item.

7.2 Items in the Notice of Nonconforming Work related to torsion during construction

Arup-CFC has prepared technical material to present to IBT to explain in more detail the nature of the torsional behavior of the bridge during construction. However, all parties agreed on July 29, 2022, to prioritize the other four primary items for the meetings that were intended to take place on August 11, 2022.

Therefore, we propose:

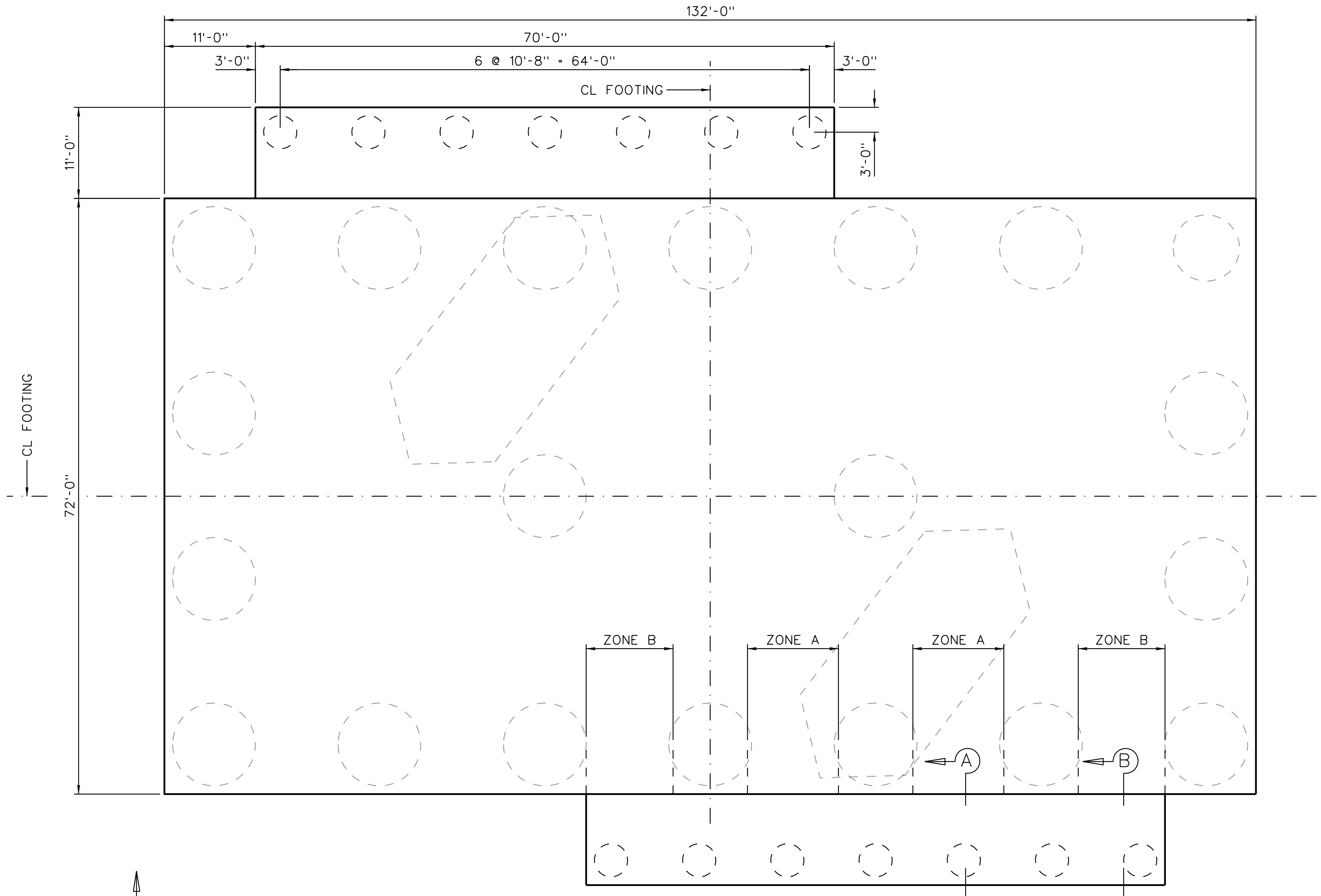
- **Action regarding Notice of Nonconforming Work Items 18 and 19 |** Schedule a meeting to explain the torsional behavior of the bridge during construction and take appropriate actions thereafter to satisfy the concerns of IBT

7.2.1 Schedule

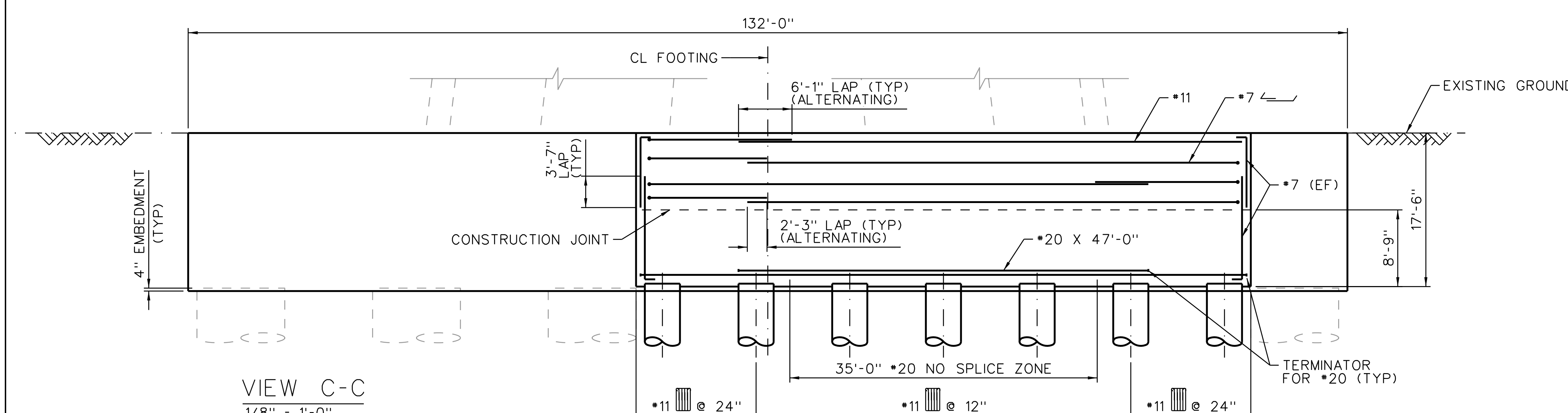
Once we understand whether any other areas of concern related to erection loading remain we will be able to propose a plan and schedule to address those concerns (if any).

Appendix A

Proposed Design Modification Tower Footing



PLAN VIEW
1/8" = 1'-0"



VIEW C-C
1/8" = 1'-0"

NOTES:

1. ALL DETAILS SHOWN ARE PRELIMINARY AND SUBJECT TO DEVELOPMENT.
2. EXTENSION TO FOOTING SHALL BE 5.5 KSI CONCRETE.
3. LENGTH OF 4FT DIAMETER DRILLED SHAFTS SHALL BE SIMILAR TO EXISTING DRILLED SHAFTS.

NOT FOR CONSTRUCTION

SCALES SHOWN FOR FULL SIZE DRAWINGS (22"x34")

NO	DATE	REVISION	APPV

FLATIRON/DRAGADOS LLC

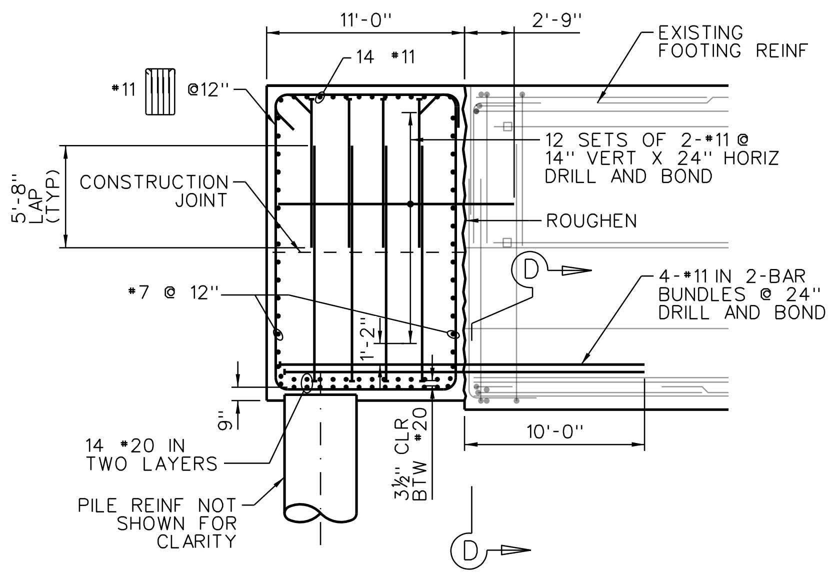
ARUP CFC

Texas Department of Transportation

US-181 HARBOR BRIDGE

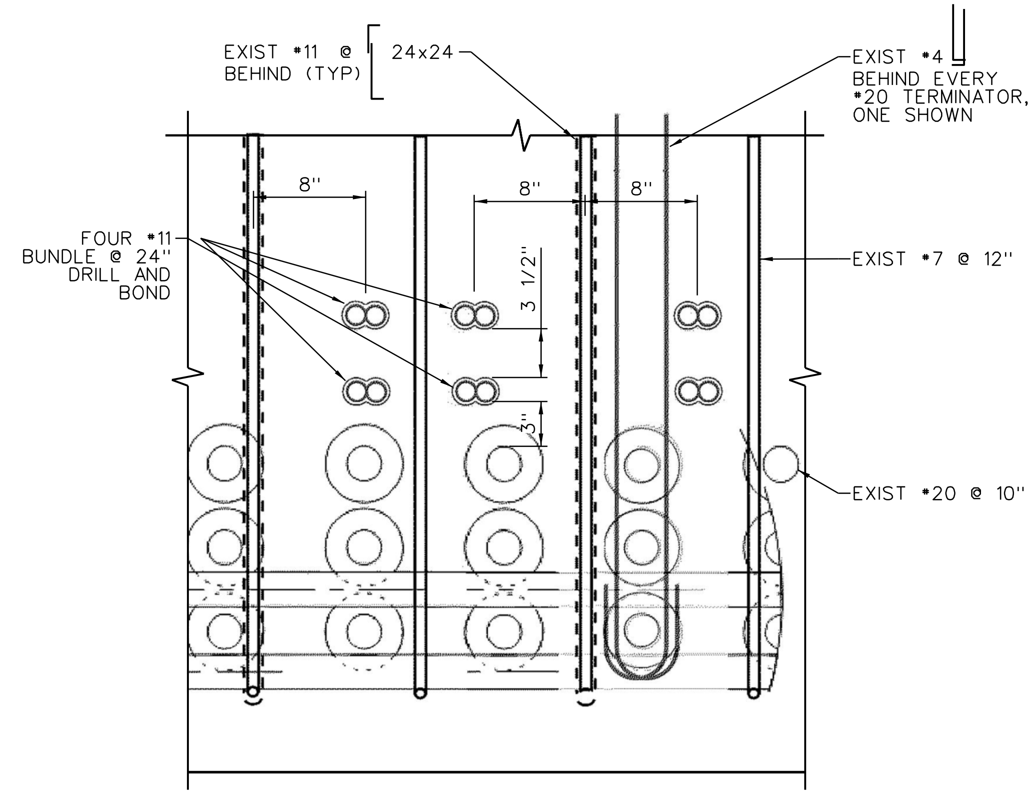
MAIN SPAN
TOWER 1NT & 1ST
FOOTING EXTENSION I

DESIGN	FED.RD. DIV.NO.	FEDERAL AID PROJECT NO.		HIGHWAY NO.
QM	X	(See Title Sheet)		US-181
GRAPHICS	STATE	DISTRICT	COUNTY	SHEET NO.
RG	TEXAS	CRP	NUECES	
CHECK	CONTROL	SECTION	JOB	
LT				SK 10
CHECK				
MC	0101	06	095	



SECTION A-A

3/16" = 1'-0"
ZONE A REINFORCEMENT

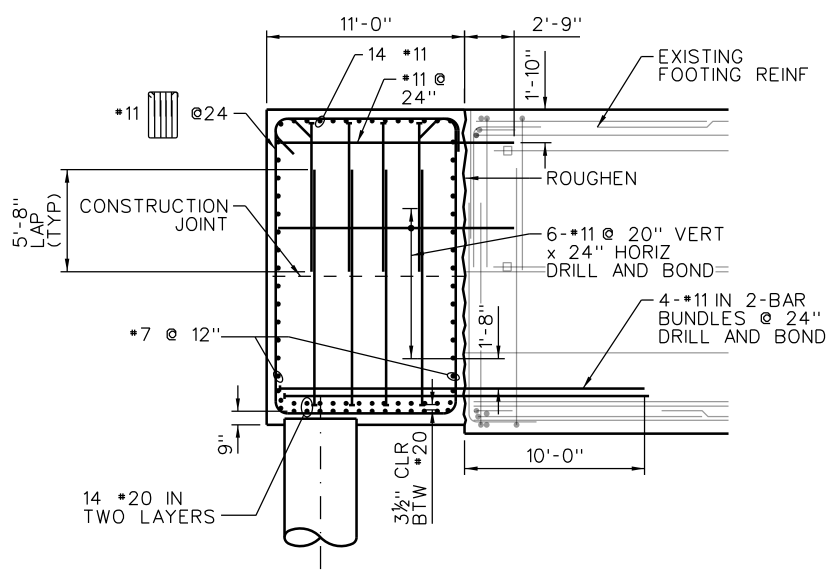


SECTION D-D

NTS

NOTES:

1. INTERFACE BETWEEN FOOTING AND EXTENSION SHALL BE INTENTIONALLY ROUGHENED TO 1/4" MINIMUM AMPLITUDE.
2. LOCATION OF SIDE REINFORCEMENT SHALL BE IDENTIFIED BY COVER METER SURVEY OR REMOVAL OF COVER.
3. LOCATION OF SHEAR REINFORCEMENT IN EXISTING FOOTING SHALL BE IDENTIFIED AND MAPPED PRIOR TO INSTALLATION OF ANY DRILL AND BOND BARS. ALL DRILL AND BOND BARS SHALL BE IN THE APPROXIMATE CENTER BETWEEN MAPPED LINES OF SHEAR REINFORCEMENT. ADJUSTMENT BASED ON AS-BUILT LOCATION OF VERTICAL REINFORCEMENT TO BE COORDINATED WITH EOR.
4. BARS WILL BE STOPPED SHORT OR OMITTED WHERE THEY WOULD OTHERWISE CLASH WITH THE TOWER DOWEL BARS.
5. #20 BAR DESIGNATIONS ARE WILLIAMS GRADE 75 THREADBAR. THEY SHALL BE SPLICED WITH COUPLERS CAPABLE OF RESISTING 125% OF THE BAR YIELD STRENGTH.
6. REINFORCEMENT FROM DRILLED SHAFT INTO FOOTING NOT SHOWN FOR CLARITY.



SECTION B-B

3/16" = 1'-0"
ZONE B REINFORCEMENT

NOT FOR CONSTRUCTION

SCALES SHOWN FOR FULL SIZE DRAWINGS (22"x34")

NO	DATE	REVISION	APRV

FLATIRON/DRAGADOS LLC

ARUP CFC

Texas Department of Transportation

US-181 HARBOR BRIDGE

MAIN SPAN
TOWER 1NT & 1ST
FOOTING EXTENSION II

DESIGN	FED.RD. DIV.NO.	FEDERAL AID PROJECT NO.		HIGHWAY NO.
QM	X	(See Title Sheet)		US-181
GRAPHICS	STATE	DISTRICT	COUNTY	SHEET NO.
RG	TEXAS	CRP	NUECES	SK 11
CHECK	CONTROL	SECTION	JOB	
LT	0101	06	095	
CHECK MC				