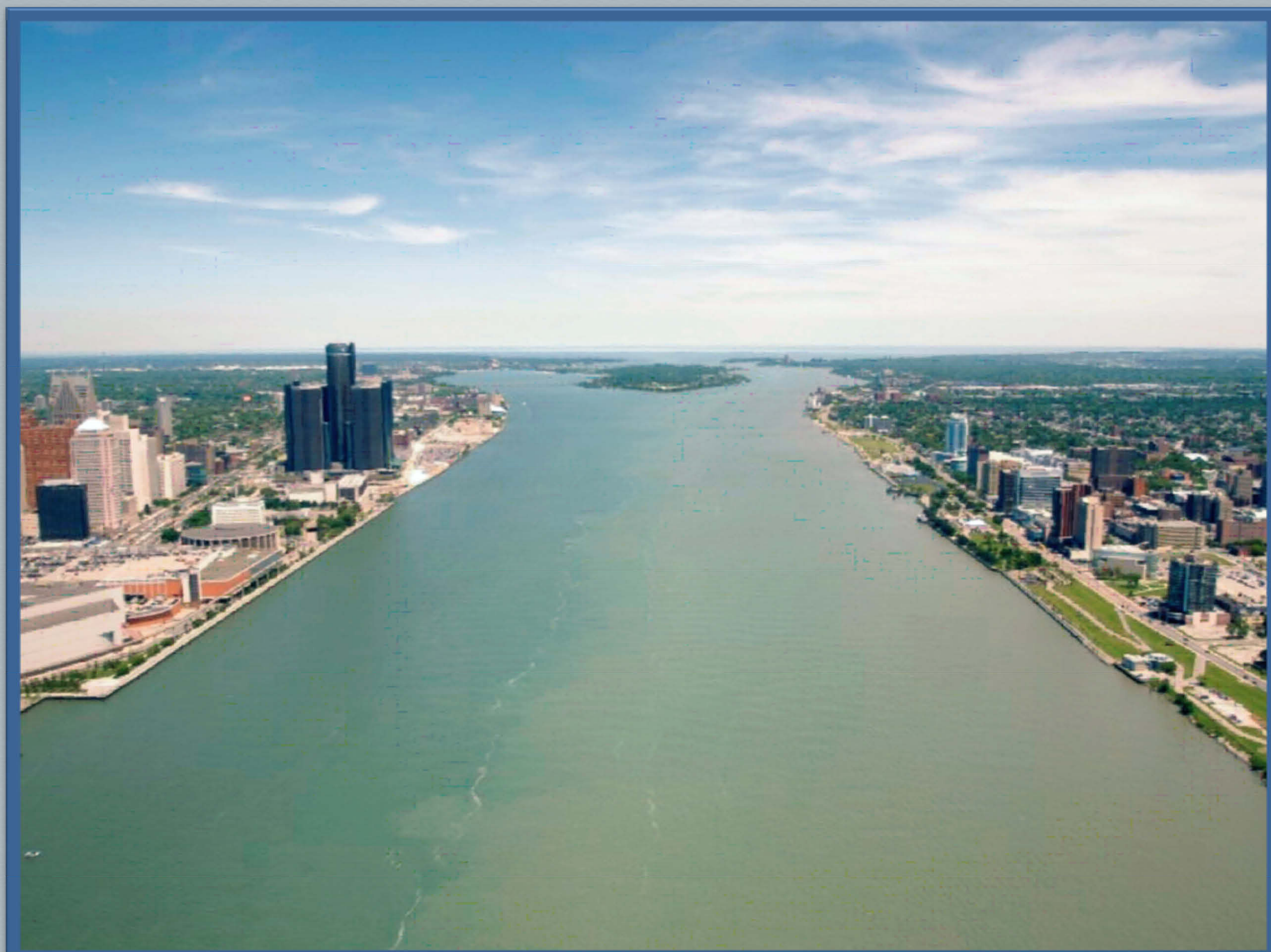
Detroit River
INTERNATIONAL CROSSING
PROJECT

A BORDER TRANSPORTATION PARTNERSHIP

U.S. Department of Transportation Federal Highway Administration | Canada | MDOT Michigan Department of Transportation | Ontario



DETROIT RIVER INTERNATIONAL CROSSING

Engineering Report

VOLUME 5: DETROIT RIVER BRIDGE STRUCTURE STUDY

November 2008

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In association with:

benesch | NCI Northwest Consultants, Inc. | NTH Consultants, Ltd | Somat Engineering, Incorporated

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1. Executive Summary

1.1. Introduction

This report documents the development of the two bridge options advanced as part of the Preferred Alternative (PA), see **Figure 1** and **Figure 2**. These bridge options represent a component of the Preferred Alternative for crossings X-10(B) (**Figure 5**).

This report was developed using the Bridge Conceptual Engineering (CE) Report, dated February 2008, authored by Parsons and URS. Generally, Parsons designed Option 7 and URS designed Option 4. The Bridge CE Report was included in the Draft Environmental Impact Statement (DEIS). The Bridge CE Report presented the development of four bridge options associated with the Practical Alternatives. The process for evaluation and elimination of the crossing alignments is presented in the body of the Engineering Report and is therefore not restated here. The conclusion of the Engineering Report analysis was that alignment X-10(B) was preferred.

A third horizontal alignment, X-10(A), was developed in the Type Study phase to avoid the area around a known sinkhole from historical brine mining in Canada if necessary. The Type Study demonstrated that Crossing X-10(A) is not preferred from a bridge engineering perspective, therefore advancing conceptual engineering of bridge options at X-10(A) was postponed until preliminary results are obtained from the geotechnical investigation program and any other relevant project EA/EIS studies. The final recommendation from the geotechnical investigation program concluded that alignment X-10(B) was feasible therefore X-10(A) was not advanced further (see the DEIS Appendix G – Summary of Geotechnical Advisory Group Activities and DEIS Technical Report – Brine Well Cavity Investigation Program, Part 1 and 2).

The scope of this Structure Study is to document the development process for the preferred alignment of the main bridge crossing the Detroit River including both the main structure over the river and the U.S. approach structure. The Canadian members of the Border Transportation Partnership will perform further development of the Canadian approach structure later. For the recommended project alternative, or Preferred Alternative, two bridge types, suspension and cable-stayed, are advanced for further development in this Early Preliminary Engineering phase.

1.2. Design Criteria

The main river crossing structure is subject to the design codes of both the U.S. and Canada and the project has been developed using the International System of Units (SI units). The design shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, SI Units, 4th Edition, and the Canadian Highway Bridge Design Code, CAN/CSA S6-06 (S6). In general the more restrictive code shall govern.

As identified in the CE phase the predominant site constraints are the required navigation envelope and horizontal alignment that avoids major industries while connecting to the Toll and Inspection Plazas in the U.S. and Canada. The bridge is designed to clear span the Detroit River with a clearance of 40.54 m at the river's edge, **Figure 7**. The third major component is the bridge cross section. **Figure 8** and **Figure 9** show the initial cross section configuration and a possible future configuration, respectively. The initial proposed cross section consists of six lanes with shoulders and a 1 m flush median. A TL-5 barrier is proposed. The bridge design condition is for future restriping to accommodate eight lanes with a median barrier, and removal of the sidewalk.

The design life, for statistical assessment of appropriate loads, is 75 years in accordance with AASHTO LRFD Bridge Design Specifications Article 1.2 – Definitions.

The service life for assessing serviceability of all components is 120 years. For specific components where it is not practicable to achieve a 120 year life, these components should be designed with the ability to be replaced.

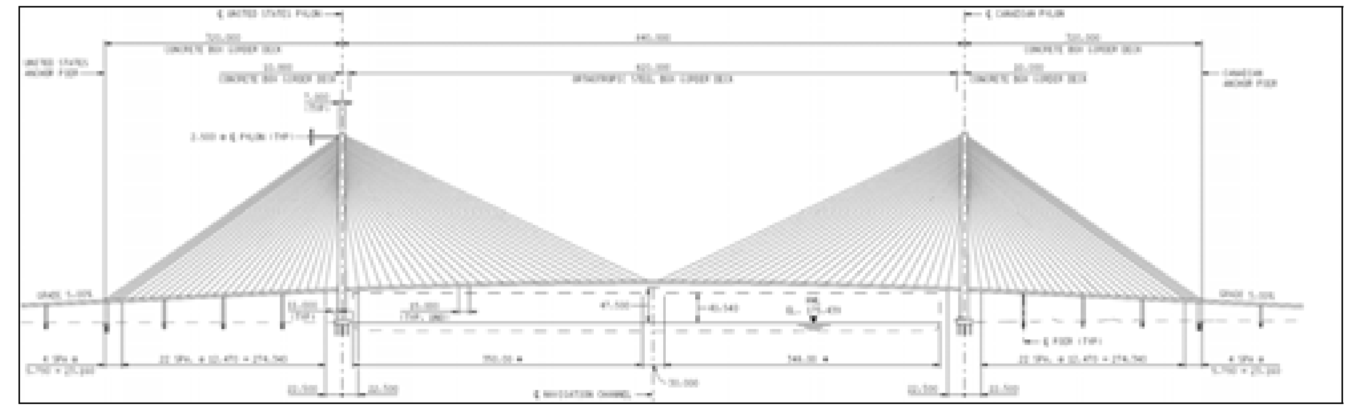


Figure 1: X-10(B) Option 4 Cable-Stay Bridge Elevation

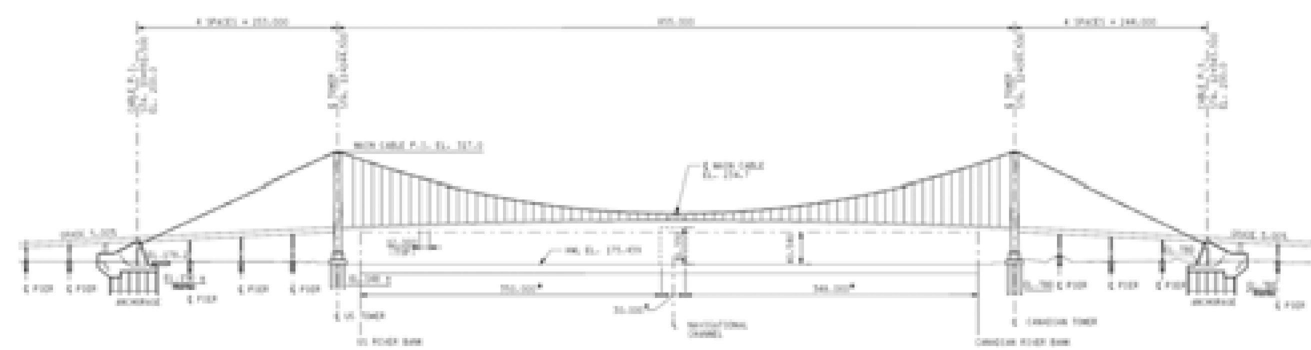


Figure 2: X-10(B) Option 7 Suspension Bridge Elevation

1.3. Cost Estimate & Schedule

The basis of the cost estimating for this report is on a unit-price type estimate. The unit price values were derived from a combination of historical unit price information from other similar projects and project specific price information from potential suppliers. This cost data was updated after the CE phase report and are presented in 2008 dollars. Inflationary factors are applied to the project as a whole and not individual components such as the bridge.

The unit prices for major items such as steel and concrete were verified with labor, equipment and material based estimates (contractor style estimate). This review focused on the large cost elements to assure that the complexities of this project, current market conditions, and the bi-national nature of the project had been properly accounted for in the unit price development.

The quantities for each of the unit price items were developed based on the level of conceptual engineering performed for the structure options. The conceptual engineering focused on the development of the principal structure member sizes (primary load path definition) based on computer analysis of the structure under a limited number of loadings that were judged as the controlling load cases. **Table 1** presents the construction cost estimates.

Table 1. *Construction Cost Estimates (in $millions).*

Crossing	X-10(B)	
Option	4	7
Main Bridge		
Bridge Construction Subtotal	441	419
General Conditions, Bond & Insurance (11%)	49	46
GC's Overhead and Profit (10%)	49	47
Design Contingency (10%)	54	51
Construction Contingency (20%)	119	113
Subtotal	**712**	**676**
Approach Bridge		
Approach Construction Subtotal	62	103
Design Contingency	9	16
Construction Contingency (20%)	14	24
Subtotal	**85**	**143**
Grand Total (Rounded)	**800**	**820**

Table 2 shows the division of costs between the U.S. and Canada with the assumption that costs of the main bridge are half and half and each approach is the responsibility of the respective country.

Table 2. *Construction Cost Estimate – By Country (in $millions).*

Option	US Cost (millions)	Canadian Cost (millions)	Total (millions) See Note 1
Option 4 - Cable-Stayed Bridge			
Approaches	35	49	83
Main Bridge	356	356	711
Total	***390***	***404***	***800***
Option 7 - Suspension Bridge			
Approaches	57	83	140
Main Bridge	338	338	676
Total	***395***	***421***	***820***

Life cycle costs include the anticipated future expenditures to maintain the bridge through its service life, 120 years, including inspections, replacement of worn out elements, and regular maintenance. **Table 3** shows the life cycle costs for each option in 2008 dollars using discount rates at 3%, 5% and 7%.

Table 3. *Life Cycle Cost Estimates (in $millions).*

Crossing:	X-10(B)	
Option:	4	7
Discount Rate	Cable-Stayed	Suspension
3%	472	514
5%	456	500
7%	450	495

A construction schedule was prepared for each bridge option following the same process used in the Bridge Conceptual Engineering Report, which is, developing a schedule based on consistent production factors for the quantities estimated. **Table 4** presents the estimated construction durations. **Appendix B** contains the detailed construction schedules.

Table 4. *Construction Durations.*

Bridge Option	Construction Duration (months)
Crossing X-10(B)	
Option 4 – Cable-Stayed	42
Option 7 – Suspension	46

1.4. Structure Study Considerations for Further Development

For the Main River Bridge several issues require additional investigation in Preliminary Design. These issues include:

- New materials
- Aerodynamic stability investigations
- Inspection access
- Durability
- Structural monitoring
- Security/hardening
- Continuation of Bridge Aesthetics and incorporation of Context Sensitive Solutions (CSS)
- Further examination of the transition from the concrete box section to the steel box section at the edge of the river for the cable-stayed bridge options.

For the U.S. approach bridge several potential refinements can be investigated during the final design stage of the project:

- Investigate providing transverse expansion capability for the deck and eliminate the longitudinal deck joint where feasible.
- Review the use of voided columns for the tall piers.
- Consider optimization of the 4-span continuous structural steel units by shortening end spans and lengthening interior spans in lieu of using constant span lengths.
- Consider using structural steel girders combined with the pre-cast concrete Michigan 1800 girders for the suspension span type main bridge. The structural steel girders would replace the post-tensioned, modified Michigan 1800 girders in the spans between the anchor block and main span tower.
- Consider providing enough width on the Canada bound side at initial construction to eliminate the need for a future widening if an eight lane section becomes justified.

2. Introduction

2.1. Project Background

The Border Transportation Partnership, consisting of the U.S. Federal Highway Administration, Transport Canada, Michigan Department of Transportation, and Ontario Ministry of Transportation, identified the need for a new or expanded crossing of the Detroit River in 2004. The planning process began with the identification of Illustrative Alternatives, consisting of the U.S. and Canadian approach roadways, toll/inspection plazas, and the international crossing structure.

Through a comprehensive technical evaluation process, with input from the public, an Area of Continued Analysis (**Figure 5**) incorporating the two crossing corridors X-10 and X-11, was identified for the development of Practical Alternatives. The bridge options have been advanced through a three-step process; Phase 1 is the Bridge Type Study (TS phase); Phase 2 is the Bridge Conceptual Engineering (CE phase); and, Phase 3 is the Early Preliminary Engineering (EPE Phase) Structure Study. This report documents the development of the two bridge type options on alignment X-10(B) advanced through as part of the Preferred Alternative (**Figure 3**and **Figure 4**).

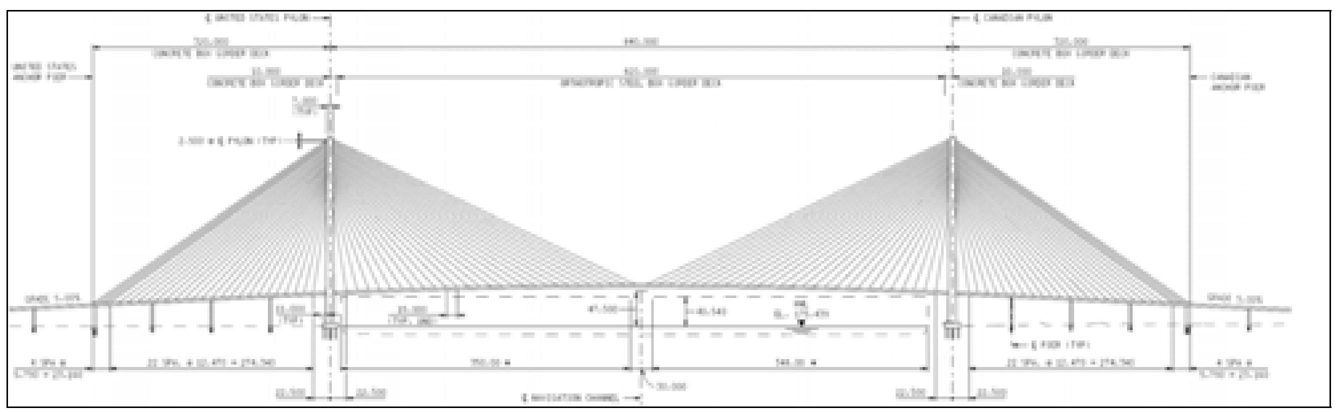


Figure 3: X-10(B) Option 4 Cable-Stay Bridge Elevation

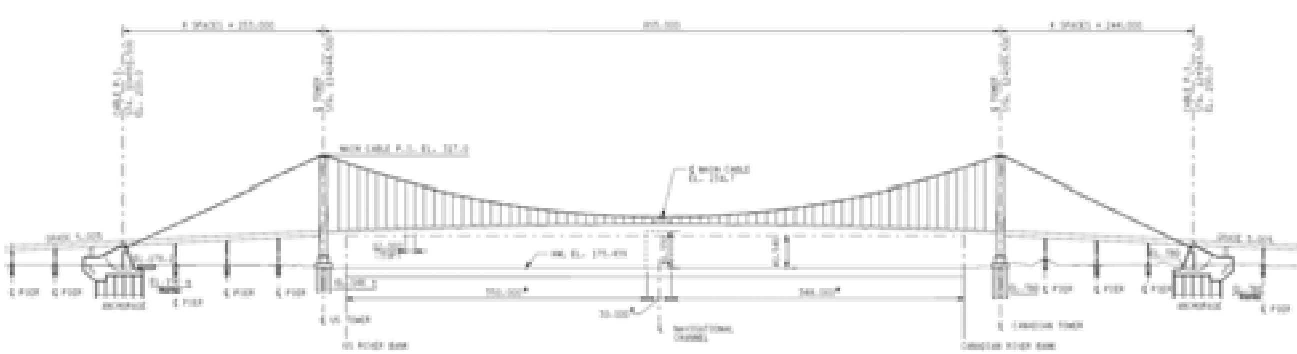


Figure 4: X-10(B) Option 7 Suspension Bridge Elevation

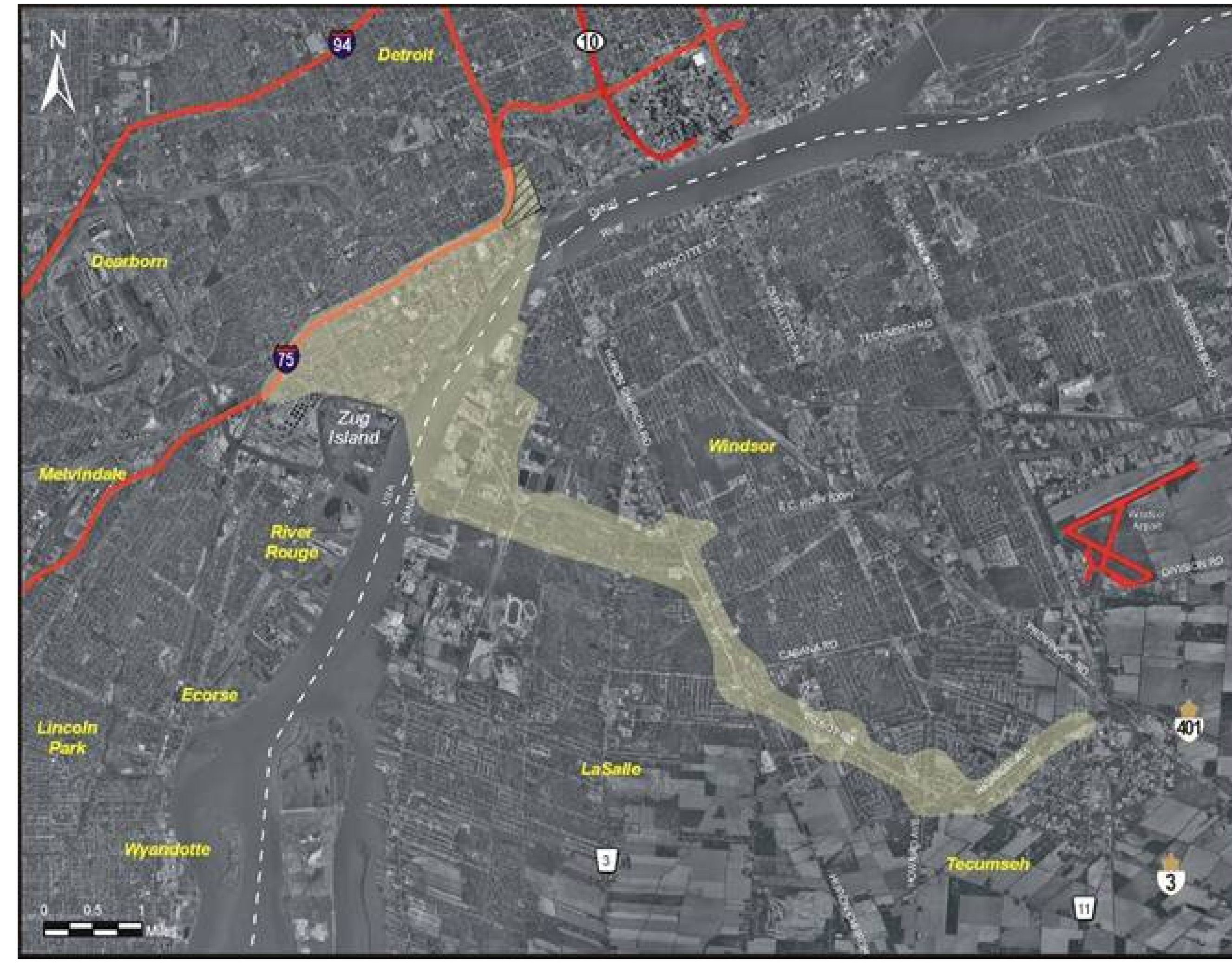


Figure 5: Area of Continued Analysis

2.2. Crossing Locations

Two crossing corridors were identified in the Illustrative Alternative phase, X-10 and X-11, which were associated with Plazas C3 and C4 in the U.S., and Plazas C2, C3, and C7 in Canada. At the beginning of the Practical Alternative phase these plaza locations were generalized into an "Area of Continued Analysis", **Figure 5**, and revised plaza locations were identified in consultation with public stakeholders and agencies. After the refinement of the plaza locations in the U.S. and Canada the X-10 and X-11 river crossing corridors were reexamined.

Based on the avoidance of major industries and cultural properties such as Brighton Beach Power Station, Mistersky Power Plant, and Fort Wayne, two horizontal alignments were developed, X-10(B) and X-11(C). A third horizontal alignment, X-10(A), was developed to avoid the area around a known sinkhole from historical brine mining in Canada if necessary. The X-10(A) alignment starts near the location of X-10(B) in the U.S. and lands in Canada south west of Brighton Beach Power Station. The three alignments are presented in **Figure 6**. Crossing X-10(A) is not preferred from a bridge engineering perspective, as detailed in the Bridge Type Study Report, therefore advancing conceptual engineering of bridge options at X-

10(A) was postponed until results were obtained from the geotechnical investigation program. The final recommendation from the geotechnical investigation program found that crossing X-10(B) was feasible therefore crossing X-10(A) was dropped from further consideration.

2.3. Bridge Alternatives

In the vicinity of corridors X-10 the Detroit River is approximately 790 m wide. Currently, major commercial shipping exists on the Detroit River as well as many shoreline industries in the project area receive delivery of goods and materials via ship. Therefore, it is necessary to provide a navigation envelope of adequate size so as not to restrict marine traffic. The options advanced from the TS phase to the CE phase included only bridges that span the entire river with a single clear span (i.e., both main towers are on the shore), based on strong objections to piers in the river from both U.S. and Canadian Lake Carriers Associations, river pilots, Transport Canada Marine Safety Division and the U.S. Coast Guard. Navigation requirements are addressed in **Section 4.1**. The X-10(B) alignment crosses the river at a skew angle of 21 degrees (skew angle measured from a line perpendicular to the centerline of channel to centerline of bridge). The combination of skew and the requirement to clear span the river result in main span lengths 840 m or longer being considered during conceptual engineering for the Detroit River crossing. At this length the only practicable bridge types are cable-stayed and suspension bridges. Main span lengths are shown in **Table 5**.

Table 5. *Preferred Alternative Main Span Lengths and Bridge Types.*

Alignment	Option	Main Span (m)	Bridge Type: Cable-Stayed (C) Suspension (S)
X-10(B)	4	840	C
	7	855	S

Note: Bridge option numbers have been carried forward from the Bridge Type Study Report.

Span lengths are approximate based on physical site conditions. During final design these may be varied to optimize the structures and respond to physical constraints such as the seawall tiebacks or utilities. In the structure study Option 4 is 15 m shorter than Option 7 primarily due to the location of the Canadian pier and its relationship to the seawall.

Table 6 shows the tower heights above approximate ground level and elevation of the tower top for each of the bridge types. The elevation is above Mean Sea Level (MSL) using the State Plane Coordinate system. It should be noted that the final tower heights could vary due to optimization of forces during design.

Table 6. *Preferred Alternative Tower/Pylon Heights*

Alignment	Option	Tower Height (m)	Tower Elevation (m)
X-10(B)	4	254.5	431
	7	139.8	317

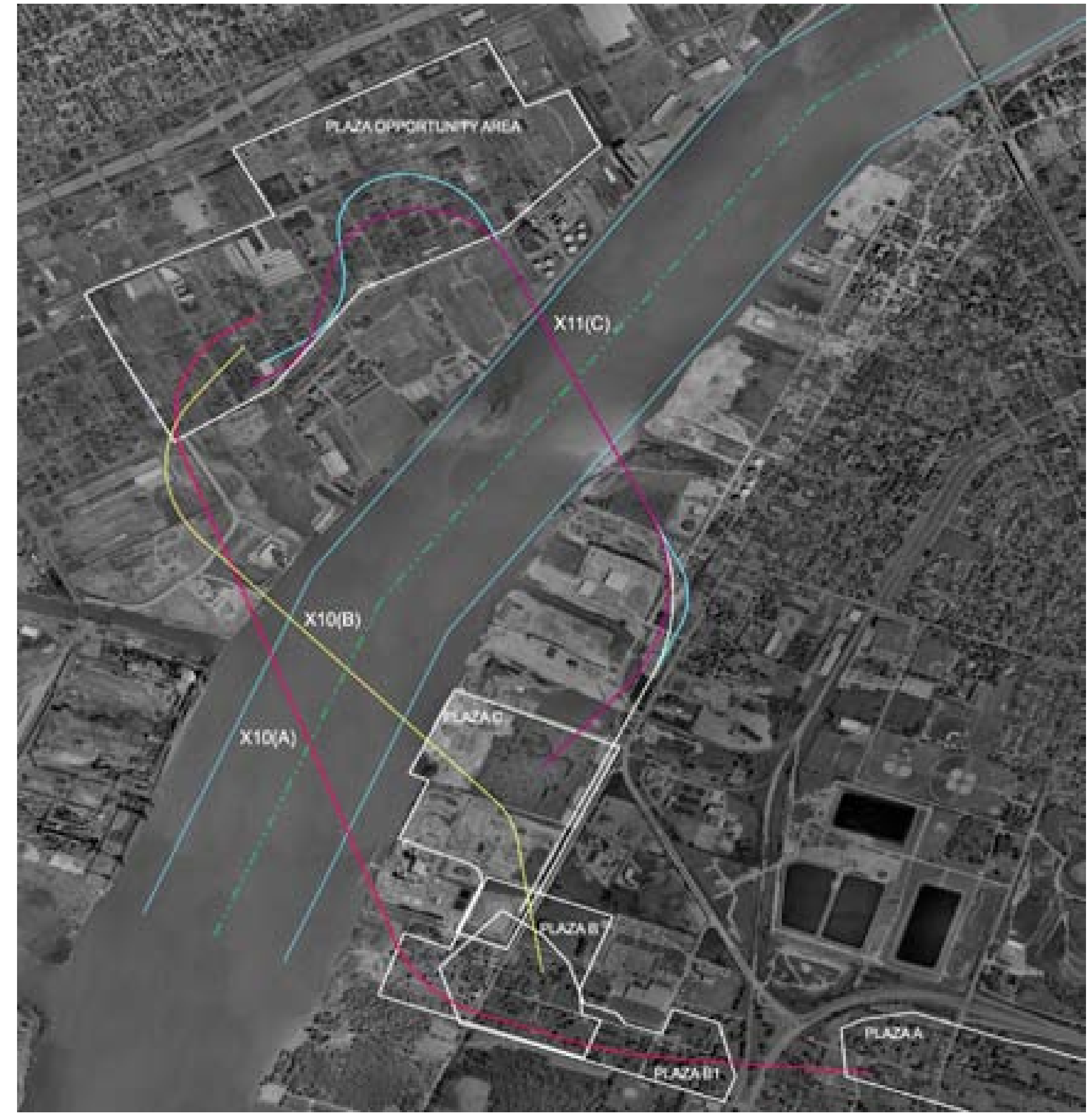


Figure 6: Crossing Alignments

3. Report Scope

The Bridge Type Study Report, dated January 2007 and revised July 2007, details the evaluation of 15 bridge types at Crossings X-10(A), X-10(B), and X-11(C). Those bridge types were evaluated and screened down to four recommended bridge types at Crossings X-10(A), X-10(B). Those bridge types were then studied further as documented in the Bridge Conceptual Engineering Report, dated November 2007, revised February 2008.

The Bridge Conceptual Engineering Report documented the development process for the main bridge crossing the Detroit River, discussed the options developed, evaluated the technical merits of those options, and provided input into the evaluation of the Practical Alternatives in the DEIS. That report did not present a study of the approach structures to the main bridge, although the associated costs were included.

After the conclusion of the DEIS public comment period, the bridge alignments were evaluated along with the Plaza and Interchange alignments, as described further in the Engineering Report, resulting in the recommendation of alignment X-10(B) as the preferred alignment. For the bridge, the selection was primarily due to the geotechnical risk associated with the Canadian approaches of Crossing X-11(C) which would add significantly to the risk, cost and schedule for that crossing.

For the recommended project alternative, or Preferred Alternative, two bridge types, suspension and cable-stayed, are advanced for further development in this report. Further development will continue into the Preliminary Engineering phase.

The Preferred Alternative (PA) phase considers the entire crossing structure (i.e., main span and approach spans). The engineering should still be considered conceptual in nature. Other project components, such as the plazas, connecting roadways, and interchanges are developed separately and are not addressed in this report.

In coordination with this technical process, a comprehensive Context Sensitive Solutions (CSS) process has been undertaken with the project stakeholders. The CSS process and results are detailed in the Engineering Report and will continue through Preliminary Engineering.

The goal of the Structure Study is to present the development of the two bridge type options advanced as part of the Preferred Alternative crossing location.

This report is divided into the following sections. Section 4 through 9.cover the main bridge – that is the bridge over the river and its back spans. Section 10 contains the U.S. approach bridge study. The approach bridge study is from the abutment at the Plaza to the anchor pier for the cable-stay bridge type and up to the main tower for the suspension bridge type. Section 11 discusses main bridge quantities and cost estimates and includes the costs for the preferred alternate for the U.S. approach bridge study. Section 12 outlines considerations for subsequent development for main bridge and U. S. approach. The Canadian Approach structures are not within the scope of this report.

4. Main Bridge Design Parameters and Approach

4.1. Geometric Development

At the current level of engineering the most significant geometric constraints governing the design of the bridge are the horizontal and vertical alignments, the positioning of piers for span arrangements, the bridge cross section, and the tower/pylon configuration and height. The navigation envelope shown in **Figure 7** provides a starting point for the vertical alignment of the alternatives and is based on consultations with the U.S. Coast Guard and Transport Canada, as well as shipping industry representatives.

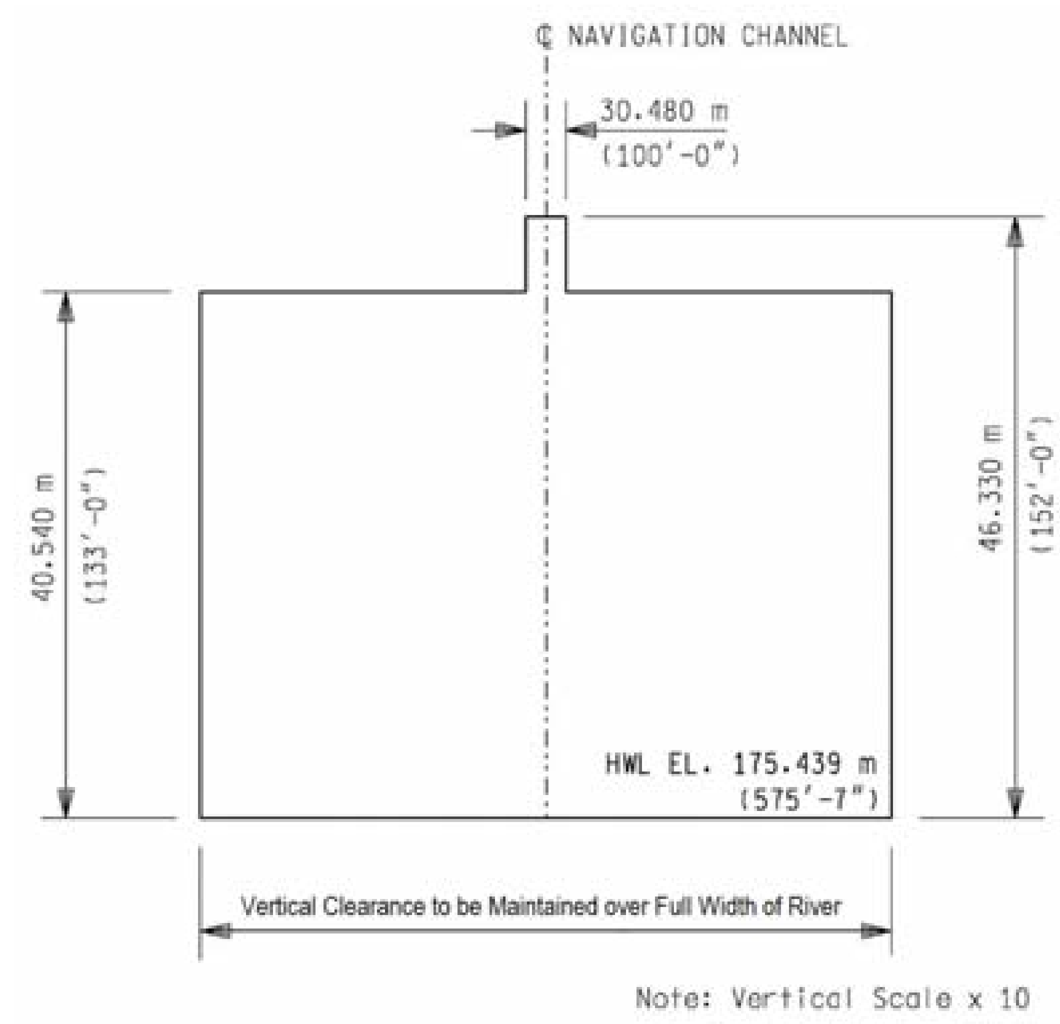


Figure 7: Navigation Envelope

Note: All dimensions shown perpendicular to the proposed channel.

The horizontal alignments have been developed in consideration of project constraints: the relative skew of the alignment to the river banks, the width of the river at the alignment location, and the requirement to clear span the river, govern the main span length and the positioning of the towers. On the Canadian side geotechnical considerations associated with historic brine well fields and the Brighton Beach power station constrain the horizontal alignment. The constrained alignment on the Canadian side, in combination with the tangent required for the structure types constrain the horizontal alignment on the U.S. side.

Side span lengths and pier locations have been advanced beyond those represented in the CE phase to improve structural efficiencies and utilize updated information to avoid known obstacles such as roadways, railroads and utilities. Suspension bridge options have been selected with unsuspended side spans, while cable-stayed options have longer suspended side spans arranged to meet site specific constraints and maintain balanced spans.

The alignments have been set to provide a tangent alignment over the entire three-span main bridge based on the longest of the side span requirements (i.e. for the cable stayed bridge options). The physical constraints in Canada noted above consequently fix the horizontal alignment location in the U.S.. In the event that a suspension bridge option is ultimately selected, the tangent portion of the alignment could be adjusted to this shorter bridge length to improve the approach alignments. However this slight adjustment is not considered a significant differentiating factor between the bridge types and results in the highest impact – the purpose of the environmental analysis.

Preliminary suspension bridge tower heights were established based on a historically efficient cable span-to-sag ratio of 10:1. The towers use inclined legs to position the cable saddles over the deck level suspender connection to produce vertical main cables and suspenders. Cross struts are placed at the tower top, below deck level, and at approximately mid-range between the two.

Preliminary cable-stayed pylon heights above the deck have been established at 20-25% of the main span length, which correlates to a historically efficient stay angle. Two pylon configurations, A-shaped and Inverted Y-shaped, were developed. Both configurations provide for two inclined cable planes originating from the top of the pylon above the center of the roadway and splaying out to the outside edge of the superstructure, adding torsional stiffness to the structure and improving vibrational behavior and aerodynamic stability.

The project design cross section is a six lane cross section, see **Figure 8**, three in each direction, with a flush median, outside shoulders and a sidewalk developed in partnership with Canada. However, as future conditions beyond the design year are not foreseeable and as modifications to a structure of this magnitude is a substantial undertaking, it is prudent to maintain flexibility in how the structure could operate in the future and take those conditions in account. For instance the addition of a median barrier in the future, say due to the elimination of Customs inspections on either side of the border and the modification of the bridge to a system-to-system free flow connector, would have a substantial dead load and aerodynamic affects. Therefore, a Future Design Allowance Cross Section, shown in **Figure 9**, has been developed which will maintain the orthotropic steel deck and cable geometry but will present a worst case load condition. This allows the bridge to operate as planned in the Proposed Cross Section and to have the flexibility to operate in other configurations up to the most severe, or controlling, condition in the Future Design Allowance Cross Section.

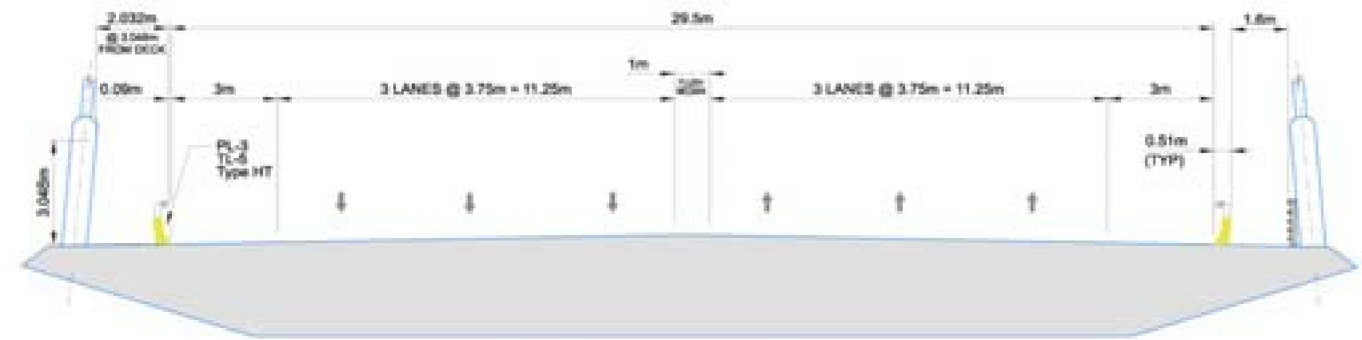


Figure 8: Proposed Main Bridge Cross Section

The bridge cross section was developed according to the roadway cross section and cable clearance. Actual cable-to-cable dimensions may vary for the cable-stayed options due to individual inclined cable geometry. The Future Design Allowance Cross Section, which represents the controlling condition, is used for design of the structure in this report.

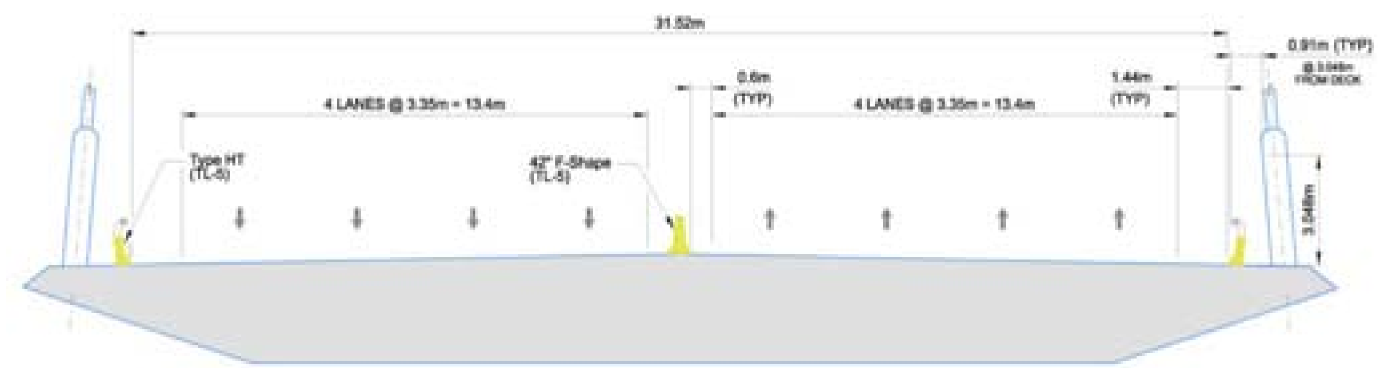


Figure 9: Future Design Allowance Main Bridge Cross Section

The suspension bridge superstructure consists of an orthotropic steel box girder in the main span with unsuspended backspans. The cable-stayed superstructure also consists of an orthotropic steel box girder for the majority of the main span. Prestressed concrete box girders are utilized near the pylons and in the side spans. The cable-stayed orthotropic steel box girder is heavier and varying in section to accommodate the compressive loads imparted by the stays.

4.2. Main Bridge Design Loads and Forces

Design loads and forces for the conceptual engineering analysis are based on the design codes of both the U.S. and Canada. Material densities/weights for common structural materials are shown in **Table 7**. The superstructure design was advanced as a steel orthotropic box girder in the main span of both bridge types and a concrete box girder in the cable-stayed side spans, and was analyzed for global loadings.

Table 7. *Weights.*

Material	Density
Reinforced Concrete	2400 kg/m^3 (150 lb/ft^3)
Structural Steel	7850 kg/m^3 (490 lb/ft^3)
Stay Cable Strand (greased and sheathed)	1.22 kg/m (0.82 lb/ft) (15.2 mm ø Seven-Wire Strand)
HDPE Stay Pipe	Varies (See **Table 10**)

The superimposed dead loads listed in **Table 8** are applied to all structure types.

Table 8. *Superimposed Dead Loads.*

Superimposed Dead Load [Item]	Unit Weight [kN/m]
Overlay	35.5
Traffic Barrier – Median	11.0
Traffic Barriers – Exterior	14.5
Traveler Rails	3.0
Lighting	0.5
Drainage	4.0
Paint	1.0
Utilities	4.0
Total	**73.5**

The current design code live loads do not apply to structures beyond 152 m (500 ft). As a result, applying AASHTO lane loadings would be overly conservative. **Table 9** reflects loading applied to the recently completed Carquinez and Tacoma Narrows Bridges and was used for the development of this report. AASHTO Table 3.6.1.1.2-1 Multiple Presence Factors should be applied as appropriate.

It is recommended that a detailed study be performed for development of final design to determine appropriate loading conditions. In addition to normal loading conditions, considerations should also be given to unique operational conditions such as multiple lane loadings for trucks, similar to what was done for the Blue Water Bridge.

Table 9. *AASHTO Lane Loads Modified for Long Spans.*

Loaded Length, L (m)	Uniform Load (kN/m/lane)	Concentrated Load at center of loaded length (kN/lane)
0 < L ≤ 185	9.34 (HL-93)	115.7 (HL-93)
185 < L < 365	11.73 – L/77.25	145.5 – L/6.21
365 ≤ L	7.01	86.7

Earthquake loadings are not considered in this phase. The low seismic zone indicates a low probability that seismic concerns will control the design other than specific components that are beyond the scope of this phase of the work.

Dynamic wind loads are likely to influence the design of specific elements of the cable-stayed option. However, due to the conceptual stage of development, only limited analysis was performed consisting of the static wind load evaluation of the tower/pylon in response to transverse winds. It is recommended that a preliminary level of wind tunnel analysis of the proposed structure and a determination of local climatology conditions should be performed in the next design phase.

Force effects from temperature were determined using LRFD Section 3.12.2, Procedure A, with a standard design temperature of 15°C and the following AASHTO Cold Climate Temperature ranges:

Steel = -35°C / +50°C (-20°C to 65°C) &

Concrete = -18°C / +27°C (-3°C to 42°C).

Other loading conditions such as Stream Flow / Scour, Vessel Collision, and Ice Accretion are not considered in this phase as none of the alternatives under consideration have marine piers. However, Ice Accretion may be considered in future phases.

4.3. Analysis

The two bridge options advanced through the this phase were analyzed using two-dimensional non-linear structural analysis computer software to determine preliminary member sizing based on the geometry, loads, forces, materials, and design criteria. The analysis consisted of a final static state analysis of the structure including dead loads, live load analysis and thermal loads. A detailed analysis of local effects in members, construction loading conditions, and dynamic effects of wind were not considered at this stage of conceptual design.

The following Strength Limit State load combinations were considered for designing the box girders, towers/pylons, stay cables, and foundations:

Strength 1a – 1.0 [1.25 DC + 1.25 DW + 1.75 LL + 1.20 TU] &

Strength 1b – 1.0 [0.90 DC + 0.90 DW + 1.75 LL – 1.20 TU], where

DC = Dead load of structural components and non-structural attachments,

DW = Dead load wearing surface and utilities,

LL = Vehicular live load, &

TU = Uniform temperature.

The STRENGTH III load combination in Table 3.4.1-1 of the AASHTO LRFD Bridge Design Specifications is considered in this study for wind loads.

The following Service Limit State combination was considered for designing the suspension bridge suspenders, main cables, and anchorages only:

Service V – 1.0 [1.0 DC + 1.0 DW + 1.0 LL + 1.0 TU].

The following live load loaded length scenarios were analyzed in the model:

1. Entire structure loaded
2. Main span only loaded
3. 50% of the main span loaded
4. Both side spans loaded

The concrete and steel box girder superstructures were designed to withstand the demands from the Strength Limit States.

Stay cable quantities were determined based on results from the Strength Limit States in conformance with the PTI Guide Specification Recommendations for Stay Cable Design, Testing and Installation, 4th Edition and Addendum 1 thereof. Stay sizes and stay pipe sizes and weights are included in **Table 10**. Steel frame anchorages in the top of the pylon were determined based on splitting and vertical forces as determined from the Strength Limit States. An assumed anchor detail at the deck level based on other cable-stayed bridge examples was used to determine quantities.

Quantities for the suspension system of the suspension bridge were determined based on results from the Service Limit State. Suspenders were designed with a Factor of Safety = 4.0 against the catalogue breaking strength. Main cables were designed to a stress level of 690 MPa (100 ksi), with a void ratio of 19% to size cable bands and saddles. Cable Bands, Saddles, and Anchor Frame sizes and quantities were determined based on a comparative evaluation of similar structures. Strand Shoes have a minimum bend radius of 230 mm and Anchor Rods were designed to ASTM A434 Class BD Material at a stress level of 345 MPa (50 ksi) on the tensile stress area (approximately $0.5F_y$).

Tower/pylon cross sections were sized and reinforcement were determined for the demands of the Strength Limit States. Reinforcement was refined on a percentage basis, based on engineering judgment and an evaluation of similar structures. Similarly, the effect of lateral loads on the cross section was based on demands from the Strength Limit States including wind, structural analysis of the tower/pylon capacity and engineering judgment considering an evaluation of similar existing structures.

Drilled shafts for the towers/pylons and anchor piers were sized based on the above Strength Limit States.

Suspension bridge anchorages have a Factor of Safety of 1.5 against overturning and sliding with at-rest soil pressures.

Table 10. *Stay Sizes and Stay Pipe HDPE Tube Sizes & Weights.*

Stay Size (number of strands)	***Outer Diameter (mm)***	***Thickness (mm)***	***Stay Pipe Weight (kg/m)***
12	*125*	*4.9*	*1.88*
19	*140*	*5.4*	*2.32*
31	*160*	*6.2*	*3.04*
37	*180*	*5.6*	*3.12*
55	*200*	*6.2*	*3.84*
61	*225*	*7.0*	*4.84*
73	*250*	*7.8*	*5.99*
91	*280*	*8.7*	*7.47*

4.4. Materials

The following materials and properties were assumed for the analysis and conceptual design of the main bridge structure components:

Reinforcing – f_y = 415 MPa (60 ksi)

Concrete Box Girder Concrete – f'_c = 45 MPa (6500 psi)

Tower/Pylon Concrete – f'_c = 45 MPa (6500 psi)

Foundation/Anchorage Concrete – f'_c = 28 MPa (4000 psi)

Structural Steel – F_y = 345 MPa (50 ksi)

Stay Cable Strand

15.2 mm ø Seven-Wire Strand (0.6 inch ø)

Ultimate Strength, f_{pu} = 1,860 MPa (270 ksi)

Strand Area = 140 mm^2 (0.217 in^2)

4.5. Main Bridge Design Criteria

The main river crossing structure is subject to the design codes of both the U.S. and Canada and the project has been developed using the International System of Units (SI units). The design shall meet the requirements of the AASHTO LRFD Bridge Design Specifications, SI Units, 4th Edition, and the Canadian Highway Bridge Design Code, CAN/CSA S6-06 (S6), and in general the more restrictive code shall govern. It should be noted that the Michigan Department of Transportation has discontinued producing or maintaining SI unit design guides, therefore, conversions will be made from U.S. Standard Units as needed.

The following documents are used in the development of the Detroit River International Crossing Conceptual Design Phase, if updated editions are available at the time of preliminary design they should be used:

AASHTO, *A Policy on Geometric Design of Highways and Streets*, 2004.

AASHTO LRFD Bridge Design Specifications, SI Units, 4th Edition and all Interim Revisions.

AASHTO, *Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals*, 4th Edition and all Interim Revisions.

Canadian Highway Bridge Design Code, CAN/CSA S6-06.

Geometric Design Standards for Ontario (GDSOH).

MDOT – Bridge Design Guide
http://mdotwas1.mdot.state.mi.us/public/design/bridgeguides/.

MDOT – Bridge Design Manual
http://mdotwas1.mdot.state.mi.us/public/design/englishbridgemanual/.

MDOT – Standard Plans
http://mdotwas1.mdot.state.mi.us/public/design/englishstandardplans/index.htm.

PTI, *Recommendations for Stay Cable Design, Testing and Installation*, 4th Edition, 2001 and 2004 Addendum 1.

The design life, for statistical assessment of appropriate loads, is assumed to be 75 years in accordance with AASHTO LRFD Bridge Design Specifications Article 1.2 – Definitions.

The service life for assessing serviceability of all components is assumed to be 120 years. For specific components where it is not practicable to achieve a 120 year life, these components should be designed with the ability to be replaced. Examples of such components include, but are not limited to: stay cables, bearings, expansion joints, deck wearing surface, navigation lighting, and roadway lighting. The bridge components requiring replacement should be identified and included in the life cycle bridge cost evaluation.

The design shall provide multiple load paths and the structure shall be continuous to achieve redundancy. Non-redundant members shall be detailed to provide internal redundancy where practicable.

The operational importance of the bridge shall be classified as "important". For seismic design purposes the bridge shall be classified as "critical".

5. Description of Alternatives

5.1. Alignment X-10(B) – Suspension Bridge Alternative (Option 7)

The suspension bridge alternate at crossing X-10(B) consists of an 855 m suspended main span and 253 m (U.S.) and 244 m (Canada) unsuspended backstay spans. The stiffening element consists of a 3.25 m deep orthotropic steel box girder. The girder is supported at 12 m intervals by wire rope suspenders connected to the main cables.

The main cables are comprised of 37 strands of 440 wires each, for a total of 16,280 galvanized 5 mm diameter (No. 6) wires. The cables are cradled in cast-steel saddles at the anchor splay and tower tops and are secured to the anchor blocks via cast-steel strand shoes.

The towers extend 141 m above their footings and are of reinforced concrete design with three post-tensioned struts connecting the legs below the roadway deck, at the tower top, and midway between, **Figure 10**. The Detroit tower is situated on land and adjacent to the river to clear the rail spur immediately south of, and servicing the LaFarge Concrete plant. The Windsor tower is sited on land within the Southwestern Sales property. The tower legs maintain a constant width for economy in forming, but vary in depth to accommodate loads that increase near the tower base. The tower legs are hollow (single cell) in cross section, allowing for access and maintenance from footing level to the uppermost strut.

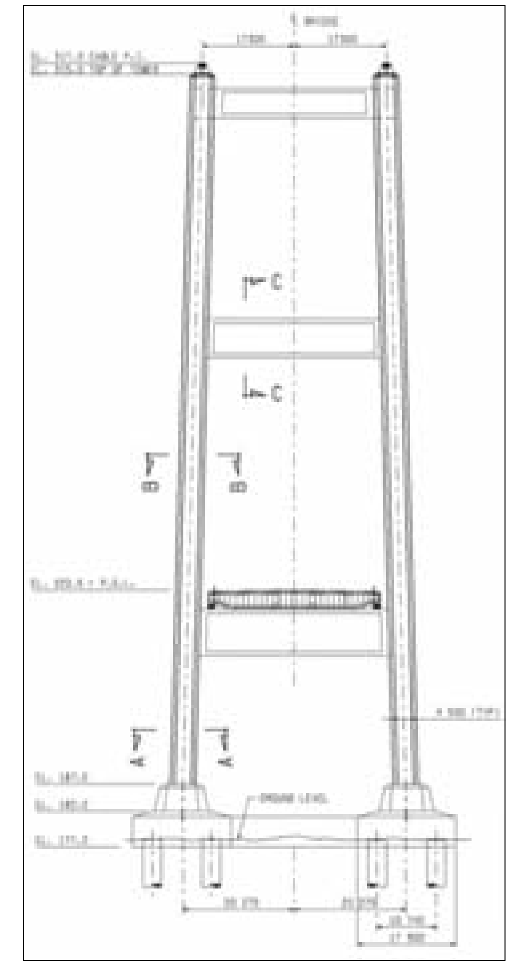


Figure 10: Suspension Bridge Tower

The gravity anchorages at each end of the bridge resist the suspension cable pull through a combination of self weight and direct load transfer to bedrock. The Detroit anchorage is situated to the north of the service road adjacent to the LaFarge Concrete plant. The Windsor anchorage has been placed in an aggregate storage facility site owned by Southwestern Sales.

5.2. Alignment X-10(B) – Cable-Stayed Bridge Alternative (Option 4)

The cable-stayed option at crossing X-10(B) consists of an 840 m main span with symmetric 320 m side spans. The side span deck and the ends of the main span deck consist of a 3.5 m cast-in-place concrete box girder, supported by stay cables and side span piers at 80 m spacing. The center 630 m of main span deck consists of a 3.5 m deep orthotropic steel box girder supported by the stay cables. The stay cable spacing in the side span is 12.5 m and in the mainspan 15 m.

The heavier concrete box girder allows the side spans to be shorter than one half the main span length. They act as counterweights when the main span is loaded with traffic, thus eliminating uplift on the anchor piers. Since there is no need to span large distances in the side spans, a continuous beam with relatively short spans is provided, which results in the side span cables acting as anchoring back stay cables. The side spans can be constructed on falsework in advance of the main span construction and will therefore provide a significant contribution to the stability of the main span construction under the free-cantilever erection conditions.

The stay cables are connected to the orthotropic steel box girder using a stay anchor weldment. The stay anchors terminate below deck level when connected to the concrete box girder. They react against a concrete block and are cast integrally with the concrete girder. At the pylon tops, stays terminate in structural steel reaction blocks cast into and integral with the pylon walls.

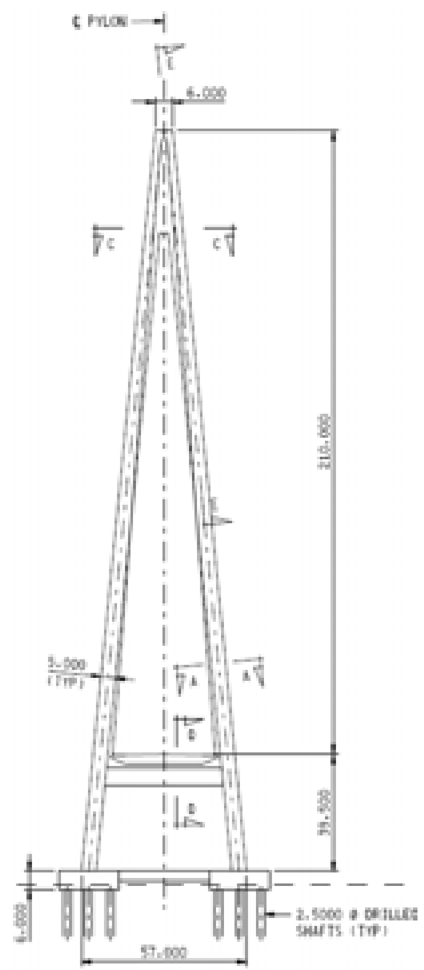


Figure 11: Cable-Stay Bridge Pylon Concepts

Two pylon configurations have been developed (**Figure 11**): an A-frame shape, as well as an inverted Y configuration. Both pylon alternatives extend 250 m above their footings, with the stays terminating within the upper 67.5 m with a stay spacing of 2.5 m in the pylon head. The pylons are of reinforced concrete design with a single cross strut below deck level. The pylon legs vary in cross section in a linear fashion simplifying forming. A hollow center is maintained, allowing for access and maintenance from footing level to the uppermost stay. The Detroit pylon is situated on land and adjacent to the river to clear the rail spur immediately south of, and servicing the LaFarge Concrete plant. The Windsor pylon is sited on land within the Southwestern Sales property.

Side span support piers consist of twin solid reinforced concrete columns with hammerhead pier caps.

6. Geotechnical Investigation and Analysis

Subsequent to the identification of the Preferred Alternative alignment additional geotechnical investigations were performed as identified in the CE Report. The purpose of this investigation was to acquire data relevant to the engineering of the main bridge foundations. A full discussion is included in Appendix D.

The subsoils encountered in the X-10 borings generally consist of variable fill soils underlain at most locations by relatively thick granular strata. Underlying the fill or granular materials is a relatively thick silty clay layer. The silty clay layer is underlain by clay or granular hardpan that extends to limestone and dolomitic limestone bedrock. The bedrock interface is generally characterized by a thin zone of low Rock Quality Designation (RQD) rock (RQD<75%), which is underlain by more competent (RQD>75%) limestone and dolomite bedrock extending to the explored depths.

Based on the results of the investigation, the existing fill deposits are highly variable and are not considered suitable for support of any foundation elements.

The underlying silty clay or granular soils are not considered suitable for support of the heavy loading expected from primary or secondary bridge foundation elements, but may be sufficient for support of ancillary structures with light-to-moderate foundation loads. For the purposes of this document, primary foundation elements are defined as the main structural foundation for cable stay and suspension bridges and the anchorages for the suspension bridge. Secondary foundation elements are defined as foundation elements for the approach roadway piers. Ancillary structures include bridge approach elements such as retaining walls, signage foundations, etc.

The hardpan soils underlying both corridors are considered well suited for the heavy foundation loading anticipated from proposed secondary structural elements of the bridge using deep foundation elements. **Table 11** summarizes the nominal pile driving resistance values for pipe piles and H-piles recommended in the Michigan Department of Transportation (MDOT) Bridge Design Manual (BDM). The dynamic resistance factor (ϕ_{DYN}) presented by the MDOT BDM is equal to 0.4, and assumes that pile driving criteria will be developed by using the Federal Highway Administration (FHWA) modified Gates Dynamic formula. If instead, American Association of State Highway and Transportation Officials (AASHTO) guidelines for dynamic testing are followed, a dynamic resistance factor of 0.65 may be used instead of 0.4 (thereby increasing the factored capacity).

Table 11. *Nominal Pile Driving Resistance.*

Pile	R_{NDR} (tonnes)
305 mm O.D. (6mm wall)	160
355 mm O.D. (8mm wall)	180
355 mm O.D. (12mm wall)	225
HP310x79	180
HP360x152	365

If a drilled pier bearing on the hardpan is used, a nominal resistance value of 3.8 MPa can be used if a settlement of approximately 5% of the shaft end diameter is acceptable. A resistance factor of 0.55 should be used with the drilled shaft geotechnical design.

The upper, highly weathered bedrock (RQD<75%) underlying the hardpan soils is generally considered suitable for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge, although bearing capacities any higher than for the hardpan (as discussed above) are not recommended.

The competent bedrock (RQD>75%) underlying the hardpan soils and the weathered bedrock is well suited for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge.

The anticipated heavy foundation loading for proposed primary foundation elements may involve drilled concrete piers . Such foundation elements would be founded on competent bedrock at least 1.5 meters into the competent limestone/dolomite bedrock. Estimated load-settlement behavior was determined for drilled pier diameters of 2.5 meters and 3.3 meters at rock socket lengths of 1.52 meters, 3.05 meters, and 4.57 meters. The ultimate nominal end resistance is approximately 28.7 MPa, while the ultimate nominal shaft side resistance in the bedrock is approximately 1.0 MPa. However, because the skin friction mobilizes at small strain, while the end resistance mobilizes at large strain, the ultimate values should not be summed to estimate the total resistance. Appendix D summarizes the computed total resistance that accounts for strain incompatibility. For the evaluation presented herein, an end resistance factor of 0.5 and a shaft side resistance factor of 0.65 are recommended, based on AASHTO and FHWA guidelines. If during final design, shaft side and end resistance values are obtained through the use of field load tests, the resistance factor for both end and shaft side resistance can be increased to 0.8.

Pipe piles to support the suspension bridge anchorage and/or main towers were also evaluated and could consist of 762mm diameter reinforced concrete filled steel pipes. The pipe piles would be pre-drilled and driven to bear on or immediately above the bedrock, a reinforcing steel cage would then be placed within each pile, and then filled with concrete. For the concept design, it can be assumed that the bedrock end bearing resistance will be mobilized within a settlement of up to 5% of the pipe diameter, which will occur primarily as elastic settlement.

The nominal pile driving resistance values for vertical and battered (3V:1H) 30-inch pipe piles is summarized in **Table 12**. The values assume plugged conditions at the pile tip.

Table 12. *Nominal Pile Driving Resistance – Pipe Piles.*

6.1.1.	R_{NDR} (tonnes)		
Pile	**Axial**	**Vertical Comp.**	**Horizontal Comp.**
762 O.D. (16mm)	900	855	285

This summary is general in nature and should not be considered apart from the entire text of Appendix D. All interpretations are for United States (US) side only and for Crossing X-10. It is also noted that the analysis and interpretations herein are with respect to the general feasibility and concept design for the bridge foundations. It is understood that the once the final design is undertaken, a more detailed geotechnical investigation and analysis will be conducted that will include additional test borings and laboratory testing.

6.2. Environmental Remediation Area

The former Detroit Coke site (between Jefferson Ave. and the Detroit River) has been identified as a significant environmental remediation area by the Michigan Department of Environmental Quality (MDEQ). From the early 1900s until 1991, coking operations were conducted at the site, which is approximately 32.4 hectares. Previous environmental investigations at the site found organic compounds (primarily benzene and naphthalene) and inorganic compounds (primarily ammonia) in the shallow fill soil and in groundwater at concentrations exceeding MDEQ Part 201 Industrial Cleanup Criteria. Dense non-aqueous phase liquid (DNAPL) exists in certain areas of the site. Some soil was also found to be characteristically hazardous for waste disposal purposes.

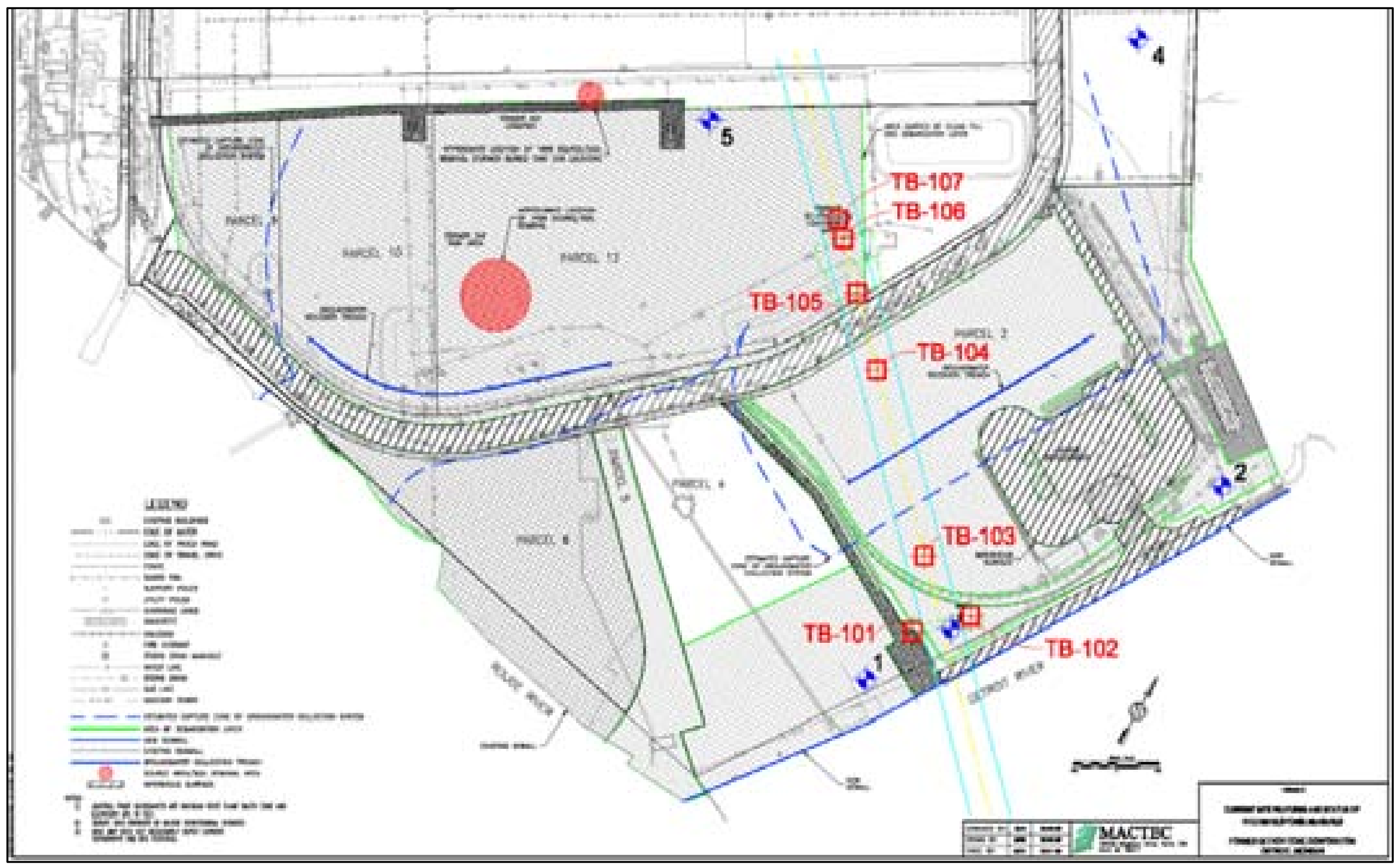
Honeywell (successor to Allied Signal) has been identified as a Potentially Responsible Party (PRP) and has installed engineering controls at the site to protect human health and the environment. An engineered barrier (imported soil cap with a geofabric layer) was installed to prevent contact with the contaminated soil and a groundwater collection trench system was installed to prevent contaminated groundwater from venting to the Detroit River. The geofabric layer beneath the imported soil cap serves as a visual indication that the soils beneath are contaminated and it serves to limit mixing of contaminated soil with cap soil. **Figure 12** shows these features as well as indicating brine well boring locations (blue) and foundation investigation borings (red).

MDOT has recently indicated that there will be restrictions on access within the right-of-way beneath and adjacent to the proposed bridge. A portion of the groundwater collection trench and some monitoring wells are currently present in the proposed right-of-way. Honeywell, under their court order administered by the MDEQ, will need to maintain or expand the groundwater collection trench, and sample the existing wells or install new wells in this area. Honeywell's access within the right-of-way will be allowed on a limited basis, it is expected that they should be able to maintain their system without significant difficulty.

Proposed conceptual modifications to typical construction methods to address the soil and groundwater contamination that will be encountered during construction are described below. These are similar to methods that both Honeywell's consultant and the MDEQ agreed to during the Brine Well Investigation Program:

Drilled Shafts (for pylons, towers, and piers):

- Install an oversized environmental casing that extends into the first clay layer. During construction, this environmental casing will prevent contaminated soil and groundwater from impacting the clean soil and groundwater below the fill. After construction, this environmental casing will prevent vertical migration since it will be socketed into a low-permeability clay. Vertical groundwater migration should also be prevented since the soft cohesive soils will squeeze against the shaft walls, which should create an effective watertight seal. As an alternative to environmental casing for individual drilled pier of pile foundation elements, a perimeter cut off wall may also be considered, that achieves the same effect as the environmental casing, but for a larger area enclosing a number of pile or pier penetrations through the upper contaminated layers.
- Dispose of spoils and used drilling mud off-site.



Source: MACTEC, 2005 & NTH Consultants, 2008.

Figure 12: Environmental Remediation Area

Open Excavation for Pile Caps over Drilled Shafts (for pylons, towers, and piers):

- The clean cap soils above the geofabric layer must be kept separate from the contaminated soils beneath. The soils originally beneath the cap can likely be reused to backfill below the cap if DNAPL is not encountered. Excess soil from beneath the cap must be disposed off site or encapsulated on site.
- Near surface groundwater must be addressed. Options include off site disposal, on site treatment and sewer discharge, or possibly discharge to Honeywell's Tar Island treatment plant, which is where the groundwater from the existing collection system is treated.
- For the pylon or tower foundation near the river, a cut off wall may be desirable to reduce the quantity of groundwater that must be addressed.

Pile Driving (for various structures):

- Prior to driving, the geofabric should be exposed and cut to prevent pulling and tearing of the geofabric.

Large Diameter Caisson (for anchorage):

- The near surface geology for the anchorage is approximately 5.5 meters of fill overlying clay. Once the clay is encountered during caisson sinking, the fill and groundwater should be removed, and the inside of the caisson cleaned. Alternatively, the construction could include pre-cutting to the clay and removing the groundwater before advancing.
- After caisson construction, vertical groundwater migration should be prevented by the soft cohesive soils squeezing against the shaft walls, which should create an effective watertight seal.
- An abandoned deep injection well is located near the proposed anchorage location. If this well is encountered during construction, measures to ensure the integrity of the well below the anchorage must be implemented.

Proposed conceptual modifications to construction methods common for all types of subsurface construction are described below. These are similar to methods that both Honeywell's consultant and the MDEQ agreed to during the Brine Well Investigation Program:

Site Restoration:

- The cap must be installed in unpaved areas. The contractor may backfill using site soil (if DNAPL is not encountered) until he is six inches below grade. The cap must then be constructed using geofabric and imported soil (or cap soil that was segregated during excavation).

Worker Health and Safety:

- To prevent skin contact with contaminated soil and groundwater, personal protective equipment should be required, including safety shoes, long pants, long sleeved shirts, safety glasses, hard hats and work gloves. If field conditions warrant, chemically resistant gloves should be worn.
- To prevent ingestion of contaminated soil and groundwater, eating, drinking, or smoking should be prohibited until after workers wash their hands.
- An air monitoring should be performed during excavation or drilling in contaminated soil.
- Hydrogen sulfide may be present at bedrock. Additional air monitoring should be conducted for this parameter during drilling or caisson construction.
- Some tasks will require employees to receive OSHA mandated HAZWOPER training.

Decontamination:

- All equipment that comes in contact with soil or groundwater below the cap must be decontaminated prior to the equipment being used on soils above the cap or prior to the equipment leaving the site.

7. Suspension Bridge Design Features

7.1. Foundations

Deep foundations are required to carry the very heavy loads from the towers and anchorage down into bedrock. Drilled large diameter concrete filled shafts are assumed. The drilled shafts extend through the upper fill, silty clay, granular soil layers, hardpan soils, and are founded into the underlying limestone bedrock formations. The competent rock layer is located approximately 30 m below the Detroit River HWL. The foundations would be designed in accordance with the parameters summarized in **Section 6** of this report, and further discussed in **Appendix D**.

7.2. Towers

The suspension bridge towers are constructed of reinforced 45 MPa concrete. Mild steel reinforcement (415 MPa) is used throughout; though higher strength steel may be used to reduce rebar congestion during final design development. The tower legs are hollow box sections.

Cross struts are hollow in cross section allowing access between tower legs. Cross struts are constructed of reinforced (415 MPa) and post-tensioned (1,860 MPa) concrete (45 MPa).

Access along the tower legs is typically provided by an elevator within one tower leg and a combination of stairs and fixed ladders in the other leg. Lighting is typically provided within the towers to light the access structures.

The tower legs rest atop solid pedestals, which in turn are fixed to a pile-supported footing. The footing is of mass concrete (28 MPa). The piles are 3.0 m diameter drilled shafts, with 16 mm thick steel stay-in-place casings. Extensive rock sockets are not anticipated, though removal of any weathered rock at the rock-soil interface may be necessary. The details for foundations for the main towers for the suspension bridge option are generally the same as for the cable stayed option discussed in **Section 8.2**.

7.3. Anchorages

Both anchorages consist of a plinth to support the splay saddles at the front face of the anchorage where both cables enter the anchorage, a splay chamber for each cable where the cable strands diverge to their respective strand shoes/anchor rods, and a mass concrete anchor block.

The anchorages are gravity-type anchorages extending to the rock and use mass concrete to resist the pull of the main cables. Multiple support structural configurations for the anchorage foundation were investigated to determine the most economical configuration. Configurations included supporting the anchorage on piles, as well as various shaped several configurations of open dredged caissons founded on bedrock similar to the nearby Ambassador Bridge. Additional configurations included various shaped open dredged caissons acting in combination with drilled shafts. In all configurations the longitudinal resistance to the cable pull is provided by direct transfer to bedrock.

Engineering efforts to date have investigated two anchorage foundation designs. The first alternative consists of two large rectangular caissons per anchorage, similar to the nearby Ambassador Bridge. The second alternative consists of a large number of vertical and battered cast-in-steel-shell piles or drilled cast in place concrete piers. The foundation elements may be keyed (or drilled) into bedrock to provide resistance to sliding.

The anchorages represent a significant portion of the cost of the suspension bridge alternatives. It is possible that with further refinement of the anchorage foundation types, additional cost reductions could be realized. Considering the cost implications of this design element, it is recommended that refinement of the anchorage be a focus in the next phase of design.

7.4. Deck System / Stiffening Element

The stiffening element is a steel orthotropic box girder. The box girder is 36.4 m wide and continuous from tower to tower. The steel skin is stiffened longitudinally with trapezoidal steel ribs welded to the steel skin. The trapezoidal ribs are hermetically sealed and pressure tested to preclude corrosion. Open (flat plate) stiffeners are used at the tips of the girder due to space constraints. The girder is also stiffened transversely with bulkheads at 6 m spacing. The bulkheads are provided with portals for access as well as chases to allow for utilities. The steel is anticipated to be 350 MPa, ASTM A709 bridge steel.

Field splices of the steel skin are welded complete joint penetration welds. Field splices of the ribs may be either welded or bolted.

7.5. Main Cables and Suspenders

The main cables are comprised of galvanized 5 mm diameter (No. 6) parallel steel wires (1620 MPa). The wires are constructed using air-spinning techniques to form 37 individual strands, which are then compacted and banded to form a circular cross section. Cast steel cable bands are clamped around the cable to maintain the shape of the cable and to receive the suspender ropes.

The suspender ropes are fabricated of galvanized, high-strength wire rope that has been prestretched and socketed with cast steel terminations. The suspender ropes and box girder are designed such that the suspender ropes at an isolated location can be removed for inspection, maintenance or replacement without closing the bridge to traffic.

Once the full weight of the bridge is hanging from the suspender ropes, the main cable wires are coated with a waterproofing paste, helically wound wrapping wire, and a three coat, highly elastic paint system for corrosion protection. The suspender ropes receive the same coating system as the main cable.

Handropes are attached above the main cable to facilitate inspection and maintenance.

8. Cable-Stayed Design Features

8.1. Foundations

Deep foundations are required to carry the very heavy loads from the pylons down into bedrock. Drilled large diameter concrete filled shafts are assumed. The drilled shafts extend through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded into the underlying limestone bedrock formations. The competent rock layer is located approximately

30 m below the Detroit River HWL. The foundations would be designed in accordance with the parameters summarized in Section 6 of this report, and further discussed in Appendix D.

8.2. Pylons

Two alternative pylon shapes were investigated. A-frame and inverted Y shaped pylons were chosen to limit second order effects and to increase the structural capacity to resist wind forces. These pylon shapes also provide additional bridge torsional rigidity.

The cable-stayed bridge pylons are constructed of reinforced 45 MPa concrete. Mild steel reinforcement (415 MPa) is used throughout. The pylon legs are hollow box sections.

A cross strut located below the deck is hollow in cross section allowing access between pylon legs. Cross struts are constructed of reinforced (415 MPa) and post-tensioned (1,860 MPa) concrete (45 MPa).

Access along the pylon legs is typically provided by an elevator within one pylon leg and a combination of stairs and fixed ladders in the other leg. Lighting is typically provided within the pylons to light the access structures.

The pylon legs rest atop a drilled shaft supported footing. The footing is of mass concrete (28 MPa). The drilled shafts are 2.5 m diameter drilled shafts, with 16 mm steel stay-in-place casings and 415 MPa reinforcing. Extensive rock sockets are not anticipated, though removal of any weathered rock at the rock-soil interface may be necessary.

8.3. Anchor Piers

There is no need to span large distances in the side spans. Therefore, a continuous beam with relatively short spans is provided which results in all the side span cables acting as anchoring back stays cables. In addition, the recommended side span superstructure is concrete to take advantage of the heavy mass to anchor the main span superstructure, which eliminates the uplift in the anchor piers. The additional side span piers will also function to stiffen the structure for live load deflections for the main span, and will contribute to stiffening the structure during the erection stage in response to wind and erection loadings.

The anchor piers are constructed of reinforced 45 MPa concrete. Mild steel reinforcement (415 MPa) is used throughout.

8.4. Deck System

The deck system has been designed to minimize wind forces on the superstructure and to provide a high torsional rigidity. This cross section can also accommodate both steel and concrete construction.

The center of the main span is a steel orthotropic box girder. The steel box girder is 35.2 m wide and continuous with the concrete box superstructure. The steel skin is stiffened and connected as described in **Section 6.3**. Outside of the center mainspan section, the deck system consists of a cast-in-place concrete box girder that is constructed of reinforced (415 MPa) and post-tensioned (1,860 MPa) concrete (45 MPa).

8.5. Stay Cables

The stays consist of 7-wire prestressing strands (1,860 MPa) protected individually with grease or wax and polyethylene sheathing. Individual stays are made up of multiple strands encased in a high density polyethylene pipe. An outer helical bead is placed on the pipe to prevent rain and wind induced vibrations.

The strands are anchored using wedges seated in an anchor head and each strand is stressed individually with a monostrand jack. Typically an additional reference strand is installed in select stays. These reference strands can be removed and inspected at a later date. It is possible to remove and replace individual strands at any point in the life of the structure.

9. Proposed Construction Methods

9.1. Suspension Bridge Alternatives

9.1.1. Tower Foundations

The tower foundations consist primarily of drilled shafts and a footing. The footing in turn consists of a pile cap at the base of each tower leg and a tie beam connecting the two pile caps.

Construction methods involve conventional techniques for drilled shafts of this size. Large diameter steel casings are drilled into the soil until they come to rest on competent rock – anticipated at approximately 30 m below existing grade. After soil and rock are excavated, prefabricated reinforcing steel cages are lowered into the casing. Reinforcing extends beyond the casing top to provide continuity with the cast-in-place footing. Cast-in-place concrete pumped into the casing completes the pile.

Of note are the limited site constraints due to the proximity of the existing LaFarge rail spur to the north and the sheet pile sea wall to the south. Also the sheet pile sea wall utilizes tie backs that would need to be addressed in Preliminary Design.

The footing (pile caps and tie beam) consist of regularly reinforced mass concrete. Both pile caps and the tie beam are cast in a single monolithic pour at each tower. With the exception of the large quantities necessary for the monolithic pour, the footing construction utilizes conventional techniques. The footing is currently shown to be entirely below grade, though this could be revisited to potentially reduce excavation costs.

9.1.2. Towers

The towers consist of three main structural elements: The tower pedestals, tower legs, and cross struts.

The tower pedestal sits atop and is reinforced to be integral with the footings. The pedestals consist of reinforced cast-in-place concrete and are solid in section. Conventional construction techniques are used.

The tower legs are hollow in section and consist of reinforced cast-in-place concrete. The towers are typically constructed using jump form technology. Reinforcing can be prefabricated off-site as much as practicable and placed by crane. Concrete can be placed by pump truck for the initial stages, though with the increasing height of later stages, concrete is typically placed by bucket, delivered by tower crane. Reinforcing congestion, particularly where reinforcing and prestressing strands from the tower struts frame into the tower legs, can be overcome with proper detailing.

As the tower legs extend higher, they may be subject to problematic wind conditions, particularly vortex shedding, which may require mitigating measures. As an example, on the recent Tacoma Narrows Bridge, a temporary steel strut was fastened between the tower legs to overcome wind vibrations.

Tower struts are hollow in cross section and are of reinforced and post-tensioned concrete. The struts are typically formed in several slabs/lifts using conventional means, though providing support can be achieved by various methods. The middle and upper struts are supported by temporary beams connected to the tower legs. The lower strut may be supported in a similar manner, or via shoring supported directly on the tower footing. Post-tensioning tendons extend through the strut walls and top and bottom slabs and are anchored to the outside face of the tower legs.

Once the towers have topped out, in preparation for receiving the main cables and deck, they are pulled back towards the anchorage, such that the weight of the main span will pull the towers back to plumb. Pull back operations consist of anchoring strands to the tower tops and tensioning the strands with tackle secured to the anchorages.

9.1.3. Anchorage Foundations

Two viable anchorage foundations have been advanced during this phase of the work.

The first anchorage foundation concept uses two 60m x 10m rectangular sunken caissons at each anchorage, one placed below each splay saddle. This approach is very similar to the nearby Ambassador Bridge. The caisson would consist of a reinforced steel cutting edge that would establish the footprint of the caisson exterior and interior walls. The cutting edge would be fabricated in sections off-site and assembled in place. With the cutting edge in place, the caisson would be "sunk" by progressively constructing the reinforced concrete walls and excavating soil from the interior. The combined effects of t he increasing weight of the caisson walls and the reduced resistance of the soil from the excavation force the caisson further into the soil. Once the cutting edge reached bedrock, it would be keyed into place and a seal slab poured. Construction of the anchorage would then begin atop the caisson walls.

The second anchorage foundation concept has been designed in a manner similar to south anchorage of the recently opened Carquinez Bridge near San Francisco, California. The foundation consists of a large number of vertical and battered large diameter pipe piles or drilled concrete piers. The vertical piles are placed below the anchor block, while the battered piles are located at the front of the anchorage and serve to resist the pull of the cables. This configuration may be optimized in future studies. The piles consist of a steel shell that extends to the top of bedrock, or may be pre-drilled and seated a nominal distance into bedrock to increase lateral capacity. Alternatively, drilled piers would be drilled under slurry, then tremie filled with concrete. Only the upper portion of the piles or drilled piers are envisioned to be reinforced, the extents of which would be identified during the next phase of the study.

9.1.4. Anchorages

Anchorage construction consists of mass concrete pours, wall construction and slab construction, all of which can be accomplished with conventional construction techniques for the respective methods. Heat generated during the mass concrete pours can often be mitigated through pour sequencing without the need of special features or operations.

Incorporated into the anchorage are anchorage points for the suspension system as well as several construction aids. These include the anchor rods and anchor frames, splay saddles, catwalk strand anchors, and tower pull back strand anchors. An access chamber is maintained at the back of the anchor frames throughout construction and in-filled after erection of the superstructure.

9.1.5. Suspension System

When the towers are complete and the anchorage construction advanced far enough to receive suspension system components, construction of the suspension system can begin. Anchor frames and grout tubes are installed in the anchorage as anchorage work progresses. Preparatory work also includes the installation of the tower and splay saddles at the tower tops and anchorages, respectively. Anchor rods are installed within the grout tubes at the anchorage splay chambers and strand shoes affixed thereto.

To provide access for cable spinning operations, a catwalk is erected from anchorage to anchorage and follows the free cable profile. The catwalk system is comprised of several support and hand strands, open mesh flooring and sides, frames at regular intervals, and several cross bridges between cables. A storm system is provided to stabilize the footwalk in high winds and provide for profile adjustment as necessary.

Custom equipment will be required at both tower tops, the splay saddles and the strand shoes to adjust the strands. A reeling plant will transfer the cable wires from coils that are delivered to the site to reels for cable spinning. The spinning equipment includes counterweight towers, drive systems, reeling plant, haul ropes, spinning wheels and all related appurtenances.

Once the catwalk and spinning equipment is in place, spinning operations begin. The main cables are constructed of galvanized high-strength steel wire, air-spun, into thirty-seven (37) strands in a hexagonal pattern formed on the point of the hex. The wire is delivered in coils from the manufacturing plant and reeled and spliced on large capacity reels for spinning operations. The wires are pulled across the span with a spinning wheel connected to haul lines suspended above the free cable profile. Each wire is looped around a semi-circular cast steel strand shoe connected to the anchor rods. Once spun, each strand is formed and bound with binding straps and individually adjusted. The hexagonal configuration of the strands of the finished cable are hydraulically compacted into a single circular bundle to receive the cast steel cable bands and later, after erection of the deck and appurtenances, coated in zinc-rich paste, wrapped and painted.

The wrapping wires are installed to a predefined minimum tension using one or more production wrapping machines. The wire is delivered in coils and must be reeled onto bobbins in the field or in a local reeling plant. The mechanically powered wrapper places the wire in a helical pattern tightly against one another for the full length between each cable band. At the bands, the wire terminates in the caulking grooves. Intermittent wire splicing for the wrapping wire is accomplished with an electric resistance butt welder.

9.1.6. Orthotropic Box Girder Fabrication

The box girder consists of a steel skin, longitudinal ribs (trapezoidal and flat plate), longitudinal bulkheads at the suspender lines and transverse bulkheads at and between each suspender location.

Trapezoidal ribs are formed to tight tolerances using a brake press and are prepared with beveled edges for an 80% partial penetration groove weld to the steel skin. Because of the large quantity of 80% penetration weld, the criticality of its performance, and the inaccessibility of the backside of the weld, the process is tightly controlled with fully automatic welding gantries and proven through prototype trials prior to production.

Ribs are welded to sections of the steel skin to create panels. The panels typically contain between 4 and 8 ribs. The panels are then joined in a pre-programmed sequence with the bulkheads to form a box girder segment.

The segments are trial assembled on the ground to the same alignment as the final position on the bridge. In this trial assembled position the field joints are prepared for field welding of the skin, bolt holes are reamed at the rib splices, geometric control points are applied to the steel, suspender pin holes are bored, and temporary construction aids are attached. The size of the segments is typically limited by transport methods and equipment used to hoist the segments into place. As an order of magnitude, it is noted that the Tacoma Narrows segments were typically on the order of 450 tonnes.

While it is understood this project is intended to have a Buy-America clause limiting procurement to North American suppliers, the following discussion is provided regarding overseas procurement as related to steel fabrication, in particular. As a matter of reference, the ongoing San Francisco-Oakland Bay Bridge (SFOBB) contains an orthotropic steel girder originally intended to be procured from U.S. suppliers. In the final analysis, however, a 400 million dollar cost savings was realized by procuring the fabricated steel from an overseas fabricator. As a matter of scale, it is noted that the SFOBB steel quantities are approximately 4 to 5 times those of the DRIC main span.

While wire, wire rope and structural strands for the suspension system may be procured at competitive prices from any number of qualified suppliers around the globe, steel fabrication of the magnitude required for the superstructure will likely be more competitively priced from offshore fabricators. In fact, the Carquinez box girder and Tacoma Narrows truss were fabricated in Japan and South Korea, respectively, and transported across the Pacific Ocean as deck cargo. The cost impact of ocean access being from the Atlantic Ocean as opposed to the Pacific Ocean has not yet been analyzed, though many similar structures have been fabricated in European countries. As an alternative method, structures of this magnitude may be fabricated off-site into panels, transported to the site and the panels assembled on-site or at a nearby assembly yard. With this method, panels could be procured from any number of sources, though with certain challenges regarding fit-up, quality control, etc.

Trial assembly would also take place at this on-site or nearby yard. This remains an option for this project.

However, not-withstanding the discussion above, the Lions Gate Bridge reconstruction (Vancouver, British Columbia) was fabricated in Vancouver and stands as an example of a major structure having been fabricated in North America, near the bridge site.

9.1.7. Orthotropic Box Girder Erection

After trial assembly, the segments are transported to the site, most likely by barge. They are hoisted into place by a pair of lifting gantries supported by, and spanning the two main cables. Lifting can be accomplished either by winches located at the tower bases with haul lines routed to the tower tops and down to the gantries, or by strand jacks mounted on the lifting gantries. In recent years, strand jacks have been the preferred option in similar situations.

Once lifted into position, the weight of the segments is transferred to the permanent suspenders. Adjacent segments are connected with temporary deck-to-deck pin connections that allow the segments to rotate with respect to each other to accommodate the changing profile of the cable as additional segments are added. As the cable becomes near fully loaded, the segments are drawn into their final relative alignment with jacking frames attached to the bottom of the box girder in preparation for the field splices. The field splices consist of complete joint penetration welds of the steel skin and longitudinal bulkheads and bolted splices for the ribs. When the splices are complete, temporary deck attachments are removed.

9.1.8. Deck Finishes

With the deck complete, operations can begin to install the electrical/mechanical systems, roadway barriers, deck water proofing, wearing surfaces, etc.

The roadway wearing surface typically consists of a two-lift, natural asphalt modified overlay, placed atop a two-layered, spray-applied acrylic membrane. Both are specialty items, requiring specialized equipment and planning. Advances have been made in recent years with the overlay that may allow forgoing the specialized equipment.

Electrical systems have successfully been installed using galvanized rigid metal conduit, fiber reinforced epoxy conduit or, alternatively, cable trays for the main runs with conduit used in the branch lines. In detailing the support system, adequate attention to expansion capabilities is important.

9.2. Cable-Stayed Alternatives

9.2.1. Pylon Foundations

As the construction methods for the suspension bridge tower foundations and the cable-stayed pylon foundations are similar, the reader is referred to **Section 9.1.1**, Tower Foundations, for construction methods of the cable-stayed pylon foundations.

9.2.2. Pylons

The pylons consist of three main structural elements: the pylon legs, cross struts, and cable anchorages. The reader is referred to **Section 9.1.2**, Towers, for construction methods of the pylon legs and cross struts, as these would be similar between structure types. In addition to the discussion in **Section 9.1.2**, the inclination of the pylon legs would necessitate temporary steel struts between the tower legs. Also, temporary tie-downs may be necessary to overcome wind forces and vibrations during construction.

The top portion of the pylon contains the anchorage zone for the stays. The resulting tensile forces from the cables in the anchorage zone can be resisted by prestressing the concrete around the anchorage or by using a steel anchor box inside the concrete pylon walls and attached through shear connectors or other means on the vertical faces. The steel anchorage box has the advantage of being fabricated in the shop in large sections containing all the supporting diaphragms and cable anchorage tubes in the correct alignment. The anchorage box sections can then be lifted into place atop the pylon, bolted together, and the remaining pylon concrete cast around the anchorage box.

It is not necessary to pull back the pylons as described for the suspension towers in **Section 9.1.2**. The vertical position of the pylons is ensured by proper sequencing of the stay cable jacking operation.

9.2.3. Anchor Piers

For the purpose of this study, the anchor pier foundations consist of drilled shafts and a footing under each pier. The construction methods involve conventional techniques for drilled shafts of this size as described above. The footing consists of regular cast-in-place reinforced concrete and is currently shown to be entirely below grade. The pier construction involves solid cast-in-place reinforced concrete columns. Multiple construction lifts and splicing of column reinforcing will be required for the tall piers.

9.2.4. Cable System

Due to the height of the pylon anchorage above the deck and the overall length and size of the cables, the conventional method of installing an entire full sized, shop-fabricated stay using a deck-mounted crane is likely not practicable.

A more likely cable installation method for a bridge of this size is the iso-tensioning method where each strand is installed and tensioned one at a time to the same force as a reference strand. The individual strands are delivered to the site on reels and the first strand is pulled from the bridge deck to the pylon top using a winch, cut to length, tensioned to a predetermined force, and temporarily anchored. The remaining strands are then pulled along the strands that have already been tensioned and are supported by temporary stirrups attached to the tensioned strands. Each strand is tensioned when it reaches the top using a small mono-strand jack with a load cell and anchored using wedges seated in the anchor head.

9.2.5. Concrete Box Girder

Outside of the center main span section, the deck system consists of a post-tensioned cast-in-place concrete box girder section cast on falsework. The construction of the side spans can be accomplished concurrent with the tower construction and can be completed in advance of the main span construction.

An alternative to casting the entire concrete box girder on falsework would be to incrementally launch the concrete box girder. This erection method utilizes stationary formwork where box girder sections are cast, cured and post-tensioned. The section is then pushed out of the formwork along the bridge alignment to clear the formwork for the next section. This construction method should be further investigated in future engineering phases to gauge its potential for cost savings.

The construction of the concrete box girder can advance independently of the pylon construction, since the concrete box girder is cast and/or incrementally launched on falsework and therefore not initially hanging from the pylon.

9.2.6. Orthotropic Box Girder Fabrication

Refer to **Section 9.1.6**, Orthotropic Box Girder, for the orthotropic box girder fabrication methods of the cable-stayed bridges.

9.2.7. Orthotropic Box Girder Erection

The center main span consists of orthotropic box girders. After trial assembly in the fabrication yard, the segments are transported to the site, most likely by barge. They would be hoisted into place from the barge, by gantries or cranes located on the bridge deck.

The steel segments are then erected in a cantilever type fashion from the edge of the completed concrete deck from both sides of the river toward the center. The first steel segment is spliced to the concrete box girder in a manner to ensure the proper transfer of loads by extending vertical interior webs of the concrete box girder into the steel deck by means of steel webs, direct bearing of the steel against the concrete, external post-tensioning of the concrete and steel sections together, and pouring a closure joint.

The field splices between the orthotropic girders consist of complete joint penetration welds of the steel skin and longitudinal bulkheads and bolted splices for the ribs.

When the splices are complete, the weight of the segments is transferred to the stay cables by jacking the stay cable. Stay cables are progressively installed and stressed in the main and side spans to balance the weight of the main span segments as they are cantilevered toward the center span closure. The center span closure is made by jacking apart the two cantilevers and installing and field splicing the center span closure segment.

A seven to ten day lifting cycle is anticipated to allow time for the complete joint penetration welding and stay stressing operations.

9.2.8. Deck Finishes

The reader is referred to **Section 9.1.8**, Deck Finishes, for deck finishes of the cable-stayed bridges.

10. U.S. Approach Bridge

10.1. Introduction

10.1.1. Project Description and Location

This section addresses structure type alternatives for the approach spans on the U.S. side of the main river bridge, connecting the roadway from the proposed toll and inspection plaza on the Detroit side with the main river span.

The overall length of the approach bridge varies with the two potential types for the main river bridge. For the suspension bridge type, the approach bridge extends all the way to the main span pier, giving an overall length of approximately 812 meters. For the cable-stay alternative, the approach span extends only to the anchor pier, prior to the back span of the main river span, giving an overall length of approximately 490 meters.

Recommendations for the approach bridge superstructure, substructure and foundation elements, including estimated costs, are presented in this section. These recommendations take into consideration the physical, economic and design constraints imposed by the site.

10.1.2. Existing Conditions

10.1.2.1. Roadway

The proposed X-10(B) alignment traverses an industrial area bordering the banks of the Detroit River, immediately north of Zug Island. The surrounding area is generally flat with large vacant areas, parking lots, paved and un-paved access roads, and railroad tracks. Current land use includes a cement terminal facility (LaFarge), major trucking terminal (Yellow Trucking), Detroit Windsor truck ferry operation, and aggregate storage (McCoy). There are residential areas north of Jefferson Avenue, which are generally intermingled with light commercial and industrial areas.

The approach bridge alignment crosses Jefferson Avenue, the Yellow Trucking facility access road, Springwells Court, and a set of railroad tracks servicing the LaFarge cement silo. With the exception of the Yellow Trucking facility access road, which will be closed, these facilities will be spanned by the bridge alternatives.

10.1.3. Geotechnical

The bedrock in the corridor is overlain by soils deposited by glacial action (glacial drift). Upper soil layers along the approach roadway and approach span alignment typically consist of very soft to soft clay deposits. The upper 3 to 6 meters of the deposits have been desiccated during historic low-water periods, resulting in soils that are very stiff to hard consistency near the surface. These upper soil layers are typically underlain by a thin layer of over-consolidated glacial till, known as “hardpan”, generally consisting of sand, silt and gravel mixed with clay. The total glacial drift thickness along the approach alignment varies from 28.8 to 30.2 meters (Elevations 147.2 to 150.6 meters), below which lies the limestone bedrock formation.

10.1.4. Utilities

The main utility impacts occur around Jefferson Avenue and Springwells Court. At Jefferson Avenue, there is an existing 300mm Sanitary line and a 120 KV power line that may be impacted by the bridge. A 305mm water line and 685mm combination line adjacent to Springwells Court may also be impacted by the bridge. There is also a sanitary line near the Yellow Trucking facility access road.

10.2. Design Considerations

10.2.1. Alignment

As described previously, the alignment identified as X-10(B) is utilized for this study. Alignment X-10(B) is initially on a tangent as it proceeds west from the main bridge span over the Detroit River. It then curves to the north with a radius of 400m as it crosses over the Yellow Trucking property and Jefferson Avenue.

The roadway cross section was established in the Bridge Conceptual Engineering Report, dated February 2008, through consultation with the Canadian Team. Three lanes in each direction, with a sidewalk on the U.S. bound side only, are initially proposed for the DRIC roadway. Consideration is given for future accommodation of 4 lanes in the U.S. bound direction with no sidewalks. The south bound side would require widening in order to accommodate four lanes, however. The roadway flares and widens at its far western end as it enters the Detroit side toll and inspection plaza, thereby accommodating the flare for the primary inspection lanes.

10.2.2. Design Criteria

10.2.2.1. Design Code:

AASHTO LRFD, 4th Edition, 2007 and supplements

10.2.2.2. Design Guide:

Michigan Department of Transportation (MDOT)

Bridge Manual and Guides

10.2.2.3. Design Live Load:

HL-93-Mod.

10.2.2.4. Weights:

Reinforced Concrete: 2402 kg/m3

Post-Tensioned Concrete: 2482 kg/ m3

Structural Steel: 7849 kg/m3

Future Wearing Surface: 1197 Pa

Stay-in-Place Forms: 718 Pa

Traffic Barriers: 707 kg/m

Pedestrian Railing (4-tube): 394 kg/m (Includes curb)

Pedestrian Parapet w/Fence: 531 kg/m

10.2.2.5. Concrete:

Cast-in-Place: Grade D, f'c = 28 Mpa

Pre-stressed: f'c = 48 Mpa (41 Mpa release)

Post-Tensioned: f'c = 48 Mpa

10.2.2.6. Structural Steel:

A709 Grade 345 Fy = 345 Mpa (Painted)

10.2.2.7. Prestressing Steel:

15.24 diameter 7-Wire Low Relaxation Strand 1862 Mpa

10.2.3. Aesthetics

Public input on aesthetics was solicited in earlier phases of the overall DRIC study. The aesthetics of the bridge substructures presented in this report are generally based on the public preferences for the cable-stayed main bridge, back-span piers.

10.2.4. Drainage

Due to the length of the approach span bridge, it is anticipated that scupper type drains will be needed to collect and remove precipitation from the bridge decks. Piping can be run externally down the pier faces to a central collection system.

10.3. Structural Systems

10.3.1. Superstructure Systems

Three types of superstructure systems are considered in this report for use on the approach bridge. They are pre-stressed concrete I-girders, structural steel plate girders, and segmental concrete box girders as discussed below. The depth of structure, including girder and slab, for these systems is limited to approximately 3m based on the current preliminary profile. Typical sections for each type are shown in **Appendix A – Figures 3-1 through 3-3**. Due to an overall width exceeding 30.5m, a longitudinal joint is required at the center of the section per the MDOT Bridge Manual.

10.3.1.1. Pre-stressed Concrete I-Girders

Based on the MDOT Bridge Manual, pre-stressed concrete I-girders are typically designed as simple spans for dead and live loads. However, the decks are detailed as continuous for live load over intermediate supports. This reduces the need for high maintenance expansion joints. In this report, four span units are typically proposed, with a continuous deck over interior supports.

Due to its efficient shape, and ability to span up to 44.2m in a simple span configuration, the Michigan 1800 girder is evaluated in this report. The Michigan 1800 girder is a wide flange girder that is 1.8m in depth. Although the MDOT Bridge Manual states that de-bonded straight pre-stressing strands are preferred, draped pre-stressing strands are required in order to optimize span length. The girders are made composite with a 0.23m cast-in-place concrete deck for live and superimposed dead loads.

The alignment of the approach bridges does have a curved section with a radius of 400m. For the longest proposed span of approximately 40.5m, straight girders can be used chorded along the curve. The largest overhang required in order to provide a minimum of 300mm from edge of deck to edge of flange is approximately 1.5m.

Although the configuration discussed above is typically used for this alternative, in the case of the suspension bridge main span type, between the anchor block and main tower, longer spans are needed to clear the railroad and for efficiency due to the significant pier heights. Post-tensioned modified Michigan 1800 girders are proposed for this purpose. The girder webs need to be modified from the typical 150mm to 203 mm in order to accommodate post-tensioning tendons. For the best fabrication efficiency, the outside profile of the girder is maintained, allowing each side of the girder forms to simply be spread apart to achieve the thickened web, thus allowing the use of existing forms.

In this system, the modified girders are pre-stressed to carry their own weight as simple spans. Once in place on the piers, a diaphragm is cast between girder ends and adjacent spans are post-tensioned together to carry the weight of the wet deck. Once the deck has hardened, the girders are post-tensioned again in a second stage to carry live load. Span lengths of up to 54.86m are proposed with this system. Due to the length of the girders, an intermediate splice might be required if shipping is impractical. This intermediate splice can be made on the ground prior to placement on the piers.

10.3.1.2. Steel Plate Girders

Structural steel plate girders are typically constructed fully continuous for dead and live load. This gives the greatest efficiency. Unlike pre-stressed concrete I-girders, steel plate girders can be curved to follow the alignment. Steel plate girders are also capable of spans in excess of 91.5m. However, spans for this study will be limited to around 61m due to structure depth limitations and curvature.

The radius of the approach span bridge is flat enough to allow the use of steel I-girders. However, the radius is tight enough to require that curvature be considered in the primary bending moments. Cross frames will also be primary members for distributing torsional forces. For this reason, cross frame spacing should be reduced over a typical straight bridge in order to minimize lateral bending stresses in the girder flanges.

Steel box girders are also a viable alternative due to their greater torsional stiffness relative to steel I-girders. However, for this radius, steel box girders typically have increased cost over I-girders due to their more complicated fabrication process. If warranted by aesthetics, steel box girders can be utilized in place of I-girders at an increased cost.

Steel I-girders are evaluated in this report. The girders are typically arranged in four span continuous units, composite with a 0.23m concrete deck. As a viaduct type structure, span lengths are typically arranged with a uniform spacing, however consideration can be given in final design to optimizing the spans for structural steel efficiency, by utilizing longer interior spans and shorter end spans for the four span units. In conformance with MDOT criteria, the girders shall be non-composite in negative moment areas.

10.3.1.3. Segmental Concrete Girders

Segmental concrete girders are typically made continuous for dead and live load, but can be detailed as simple spans with continuous deck, as with pre-stressed I-girders, when using the span-by-span method of construction. This is called a link slab or semi-continuous system. This system is discussed in the document in **Appendix A**. The semi-continuous joint replaces the full depth closure pour typically used in span-by-span construction and gives the following advantages:

- Continuity tendons, couplers and overlapping tendons are eliminated
- Continuity induced thermal effects are eliminated
- All pier segments are the same
- Unlike a full depth continuity closure pour, the semi-continuous joint is off the critical construction path
- Contractor does not have to adhere to a direction of construction set by the designer
- Design and construction are simplified resulting in reduced cost

The span-by-span method does require an overhead or under slung truss or gantry for construction of each span, which can be a large capital outlay. An overhead truss is required for areas where vertical clearance is limited. In the span-by-span system, the truss or gantry supports the segments for the entire span until they are post-tensioned together all at once. In addition, due to the need to have a truss support system, the span-by-span method of construction is most suited to viaduct type structures that have repetitive span lengths in the 30.5m to 50m range and limited structure depth (per AASHTO optimal segmental girder depth is between L/18 and L/20). Post-tensioning strands are placed inside the box void, external to the webs and flanges, allowing for thinner sections and better inspection access to tendons over other segmental systems.

Balanced cantilever and progressive cantilever construction are also possible alternatives for this structure. However, balanced cantilever structures require shortened spans at their ends, additional concrete due to internal tendons, and result in un-balanced construction loads to the piers. In addition, the loads during construction are significantly different than the final loads and require various tendon placement stages to accommodate. Progressive cantilever construction reduces some of the issues related to balanced cantilever by employing a movable temporary stay arrangement to support the cantilever sections. However, thicker sections are still needed for internal tendons. Either of these types of segmental construction can more easily accommodate longer spans than span-by-span construction. However, the spans proposed for this structure are in the range of span-by-span construction limitations.

For the purposes of this report, the span-by-span link slab system is proposed. Due to the width of the structure, two single cell segmental sections are proposed to be placed side by side to form the overall section. For a single cell box to be economical, the width of slab should be less than or equal to 6 times the depth. Therefore, each single cell section is proposed to be approximately 17.1m wide and 3m deep **(See Appendix A – Figure 3-4)**. This is near the upper limit for an economical single cell box. Where the roadway flares at the toll plaza, four single cell sections are proposed. The overhangs for these sections will vary, allowing a tapered deck to be accommodated.

10.3.2. Substructure/Foundation Systems

For support of the beam alternatives, in the initial taller part of the approach bridge, which is a much as 30.5m above existing ground, piers similar to those planned for the back span of the cable-stay alternative are proposed. These taller piers consist of two large rectangular columns with hammerhead type pier caps **(See Appendix A – Figure 3-5)**. The pier caps are continuous across the full width of deck to help create a stiffer frame. A taper is proposed in the taller columns for additional stability.

Piers for support of the segmental alternative are similar. However, a cap at the top is not needed due to the box geometry. Therefore, the piers are essentially single flared columns **(See Appendix A – Figure 3-6)**. However, a strut is added at an intermediate height in the taller piers.

As pier heights lower, the tapers and struts are eliminated. A full height cantilever abutment is proposed at the bridge end.

For support of the bridge substructure, deep foundation elements are recommended **(See Appendix D)**. Two basic types of deep foundation elements are recommended in the report as being viable for support of the approach bridge: driven piles and drilled shafts (caissons), also referred to as drilled concrete piers.

Two types of piles were evaluated in the geotechnical report: steel pipe piles and steel H-piles. The H-piles were determined to be potentially problematic due to the potential for pile “sweep”, and the inability to detect such sweep (and associated reduction in pile capacity). Pipe piles can be pre-drilled to within 1.5m of the “hardpan” layer, driven to bear within the “hardpan”, and then filled with concrete. Pre-drilling is not generally practical or cost-effective with the H-piles; and they would tend to “sweep” over the relatively deep driving distance and when hitting the "hardpan" layer. Therefore, pipe piles will be the only driven pile alternative considered in this report. Three pipe pile sizes for the approach bridge foundations were discussed in the geotechnical report (Appendix D). The nominal driving resistance values recommended in the MDOT BDM were summarized in **Table 11**.

Drilled shafts are also considered viable for support of the approach bridge foundations. The geotechnical report recommends that for supporting approach bridge foundations using drilled concrete piers, the piers should be founded at least 0.6 meters into the hardpan soils, resulting in depths of approximately 27 to 30 meters. The drilled pier should be designed for end bearing in the hardpan. For a drilled shaft constructed in this fashion, the nominal end resistance should be approximately 3.8 MPa for conceptual design purposes, which corresponds to a settlement of approximately 5 percent of the shaft end diameter. A resistance factor of 0.55 should be used. Drilled shafts should be placed with a clear spacing (edge to edge) of 3 diameters in order to avoid capacity reductions from group effects. The various proposed shaft and footing configurations are shown in a table on **Appendix A – Figures 3-5** and **3-6**.

10.4. Bridge Type Study

10.4.1. Bridge Layout

For each of the three superstructure type alternatives, two alternate configurations are evaluated, one each for the suspension bridge and cable-stay bridge main span alternatives. For the suspension bridge, the approach bridge extends beyond the cable anchorage all the way to the tower pier of the main span. For the cable-stay bridge, the approach bridge stops at the anchor pier, before the back span of the main span. Plan and elevation views of the layout for these alternatives are shown in **Appendix A – Figures 4-1 through 4-6.**

Typical sections of the alternatives are shown in **Appendix A – Figures 3-1 through 3-3**. The typical sections are configured to align with the main span. Initially, the section consists of three lanes in each direction, with a 1m flush median between travel directions and 3m outside shoulders. A sidewalk is only currently required on the U.S. bound approach to the toll and inspection plaza. This sidewalk is separated from the shoulder with a traffic barrier. A 1.066m metal railing provides fall protection on the outside of the sidewalk. The Canada bound approach does not have a sidewalk; therefore, only a traffic barrier is placed on the outside and the section width is reduced on that side.

In preparation for future traffic increases, the typical section on the U.S. bound side will allow for support of four lanes of traffic by eliminating the sidewalk, providing a median barrier, and utilizing a reduced outside shoulder. However, the Canada bound side would require future widening. It would be of economic benefit to provide the needed width in the current construction rather than attempting a future widening if a four lane section is considered a necessity in the future, and this should be addressed in final design.

Due to the overall width of the typical section, a longitudinal joint is required by MDOT criteria in the center of the median, essentially creating two sections forming the overall width. Since the overall width is just above the 30.48m criteria, consideration should be given in final design to allowing for transverse expansion. This would enable elimination of the longitudinal joint for most of the approach bridge, giving a more efficient section.

10.4.2. Alternative A – Pre-Cast Concrete I-Beams

Span lengths for this alternative were typically confined to 44.2m based on the limitations of the Michigan 1800 girder as well as overhang restrictions for chording the girders along the curved alignment. The exception to this is in the tangent area between the anchor block and main tower of the alternative for the suspension span main bridge. In that case, longer post-tensioned spans of up to 54.86m using modified Michigan 1800 beams, as discussed previously, are needed to span the railroad. The proposed typical section shown in **Appendix A – Figure 3-1** consists of thirteen girders composite with a 0.23m deck.

At the beginning of the bridge, a skewed pier arrangement is required in order to span Jefferson Avenue. A horizontal clearance of 2.75m from the edge of roadway is proposed. This clearance assumes the use of guardrail to protect the pier columns. After the first few spans impacted by the Jefferson Avenue skew, the remaining spans are all square.

The end of the approach bridge varies, based on whether the suspension or cable-stay main span bridge is assumed. Alternative A1 represents the case of the suspension bridge and Alternative A2 represents the case of the cable-stay bridge. For Alternative A1, the approach bridge ends at approximately station 11+244.50 at the main bridge tower. The approach bridge crosses over the anchorage block for the main suspension cables. Supports are provided on the anchorage block for the approach beams. The costs of the anchorage block and supports are not included in this report. The total overall length of this alternative is 811.65m, with a total of 20 spans.

For Alternative A2, the approach bridge shortens to station 10+923, which is the location of the anchor pier for the cable-stay alternative. This reduces the overall length to 490.15m, with a total of 14 spans.

As discussed earlier, the proposed typical piers are double hammerhead caps with two columns **(See Appendix A – Figure 3-5)**. The columns are assumed to be solid sections. However, in final design, hollow sections should be considered due to the height of the piers.

1.2m) diameter drilled shafts are proposed for support of the piers. Based on an analysis, the number of 1780 kN, 40.6cm pipe piles needed exceeds the cost of using drilled shafts. With the need to pre-drill and fill the piles with concrete, drilled shafts become more efficient and also minimize the foundation footprint. A cantilever abutment is proposed at the beginning of the bridge due to the relatively short end height. Abutment height has been kept to less than 5m.

The overall cost respectively for Alternative A1 & A2 is $52.03 million and $30.94 million, inclusive of mobilization and design contingences. Quantity and cost calculations are included in **Appendix C**.

10.4.3. Alternative B – Structural Steel Plate Girders

In spanning between the anchor block and tower of the suspension alternative, and when arranged in two span continuous units, span lengths of approximately 61m allow the minimization of the tall piers in this area. Therefore, this span length was generally used throughout this alternative.

The proposed typical section shown in **Appendix A – Figure 3-2** consists of six girders for each half width spaced at 3.05m. A constant web depth of 2.1m with 16mm thickness, transversely stiffened as needed, is proposed.

Due to the longer span lengths used, skewed piers are not required at the Jefferson Avenue crossing; however, the horizontal clearance of 2.75m from the edge of roadway is proposed in order to keep the span reasonable. This clearance assumes the use of guardrail to protect the substructure.

As with Alternative A, Alternative B1 represents the case of the suspension bridge and Alternative B2 represents the case of the cable-stay bridge. The total overall length of Alternative B1 is 797.83m, with a total of 13 Spans. The total overall length of Alternative B2 is 476.33m with a total of 8 Spans.

Double hammerhead piers with a similar configuration to Alternative A are used here as well **(See Appendix A – Figure 3-5)**. A cantilever abutment is again proposed at the beginning of the bridge.

The overall cost respectively for Alternative B1 & B2 is $62.90 million and $37.99 million, inclusive of mobilization and design contingences. Quantity and cost calculations are included in **Appendix C**.

10.4.4. Alternative C – Segmental Concrete Girders

The segmental alternative span length is typically limited to approximately 48.75m, which is near the practical limit for a span-by-span erection truss. Segment lengths are proposed at approximately 3m. As much as possible, the spans have been arranged in uniform increments of the segment length. For ease of construction, simple span units with continuous decks are typically assumed. However, for the final span adjacent to the main span tower of the suspension type bridge, a longer span is needed to clear the railroad. This span is on the order of 60m. At this location, a temporary support structure for the erection truss or gantry, mounted off of the main span tower, may be an alternative for placing the segments in this longer span. This final span would become part of a four span continuous unit composed of the last four spans.

The proposed typical section shown in **Appendix A – Figure 3-3** consists of two trapezoidal box sections separated by a 25mm open joint. The proposed box sections are 3m deep. Both longitudinal and transverse post tensioning are required.

At the beginning of the approach bridge in the area over Jefferson Avenue, in order to accommodate the flaring roadway width coming from the toll plaza, a second smaller segment is combined with the typical segment to form the overall width as shown in **Appendix A – Figure 4-7**. The overhangs of the segments are blocked out as needed to match the changing deck width. Piers are staggered on each half-width of the deck in order to achieve a reasonable span over Jefferson Avenue. A horizontal clearance of 2.75m and guardrail to protect the substructure is again assumed.

As with Alternatives A and B, Alternative C1 represents the case of the suspension bridge and Alternative C2 represents the case of the cable-stay bridge. The total overall length of Alternative C1 is 820.43m, with a total of 18 Spans. The total overall length of Alternative C2 is 498.93m with a total of 12 Spans.

Single column piers conforming in width to the bottom box flange are proposed for support of the box girders (**See Appendix A – Figure 3-6**). For the taller piers, a strut is proposed between the piers for stability. A cantilever abutment is again proposed at the beginning of the bridge.

The overall cost respectively for Alternative C1 & C2 is $66.56 million and $44.94 million, inclusive of mobilization and design contingences. Quantity and cost calculations are included in **Appendix C**.

10.5. Recommendations

Cost calculations are included in **Appendix C**. **Table 13Error! Reference source not found.** summarizes the costs for each structural alternative:

Table 13. *U.S. Approach Bridge Type Costs*

Alternative	Structure Type	Main Bridge Type	Cost
A1	Pre-Cast Concrete I-Beams	Suspension	$52.03 Million
A2	Pre-Cast Concrete I-Beams	Cable-Stay	$30.94 Million
B1	Structural Steel Plate Girders	Suspension	$62.90 Million
B2	Structural Steel Plate Girders	Cable-Stay	$37.99 Million
C1	Segmental Concrete Girders	Suspension	$66.56 Million
C2	Segmental Concrete Girders	Cable-Stay	$44.94 Million

Note: Costs are inclusive of mobilization and design contingences

Based on cost, Alternative A, Pre-Cast Concrete I-Girders using Michigan 1800 beams, is the most economical and is therefore recommended. This type of structure will also be relatively low maintenance compared with structural steel. In addition, the conventional construction may allow for more bidding competition as compared with the segmental alternative.

11. Quantity and Cost Estimates

11.1. Cost Estimate Basis and Assumptions

The basis of cost estimates for the main bridge portion (suspension and cable-stay) of the Structure Study is on a unit-price type estimate. The unit prices reflect the manner in which large construction projects are typically bid, and include all costs related to that particular item such as material costs, fabrication/labor costs, transportation costs, erection costs, testing/inspection/QA costs, etc. These costs therefore represent a rolled-up summary of a large number of cost items related to a particular element of construction, and therefore require some judgment in using historical unit price values to account for differences between projects. Unit price values were derived from a combination of historical unit price information from other similar projects and project specific price information from potential suppliers. Additional assumptions are listed in **Appendix C**.

The unit prices for major items such as steel and concrete were verified with labor, equipment and material based estimates (contractor style estimate). This review focused on the large cost elements to assure that the complexities of this project, current market conditions, and the bi-national nature of the project had been properly accounted for in the unit price development.

All unit prices are presented in 2008 construction dollars and represent an assessment of current market conditions for historically volatile cost elements such as steel and concrete. The cost escalation to year of construction is addressed as a separate adjustment to the final project estimate.

The estimates for the U.S. Approach Bridge were developed consistent with the Interchange Structure Study. That is, a unit price based estimate with unit costs compiled from the MDOT "Weighted Average Item Price Cost Report" including costs through the third quarter of 2008 for the Metro Region. Some costs, such as substructure concrete, were scaled up to account for the scale of the structure. The quantities for each of the unit price items were developed based on the level of conceptual engineering performed for the structure options. The U.S. Approach Bridge cost included in this section are for the recommended Alternate A, Pre-Cast Concrete I-Girders using Michigan 1800 beams, from Section 10.

The estimates for the Canadian Approach Bridge were based on a square foot estimate as developed for the Bridge Conceptual Engineering Report. They were not updated for this report.

11.2. Environmental Remediation Costs

For piers within the identified environmental remediation area (**Figure 12**) a line item cost is added to the estimate. This opinion of cost is based on known conditions and permit requirements and could vary substantially once preliminary design commences.

It is estimated that the additional costs to address the environmental issues during construction are, on an order of magnitude basis:

- For the suspension bridge option, conceptual costs to address environmental issues for the anchorage are on the order of $1,000,000. Conceptual costs to address environmental issues for the tower foundation are on the order of $3,000,000. This cost estimate assumes that an earth retention system would reduce groundwater infiltration and that the portion of the drilled shaft cap that is below the groundwater table could be constructed in 60 days.
- For the cable-stay bridge option, conceptual costs to address environmental issues for the pylon foundations are on the order of $2,000,000. It is likely that the costs for this option will be lower than for the suspension bridge option. This is because the cap over the drilled shafts for the suspension bridge option will extend much deeper, and will be closer to the river, requiring the contractor to address significant quantities of contaminated groundwater.
- For both options, the environmental cost of each approach pier is on the order of $150,000.

11.3. Initial Construction Cost

A summary of the conceptual engineering initial construction costs for the two bridge options is shown in **Table 14** below. A detailed estimate for each main bridge option is included in **Appendix C**.

These cost estimates include the main bridge over the Detroit River and the associated bridge approach cost from touch-down to touch-down points. The Canadian approach bridge costs have not been updated therefore the CE Report costs are used.

Table 14. *Construction Cost Estimates (in $millions)*

Crossing	**X-10(B)**	
Option	**4**	**7**
<u>Main Bridge</u>		
Bridge Construction Subtotal	441	419
General Conditions, Bond & Insurance (11%)	49	46
GC's Overhead and Profit (10%)	49	47
Design Contingency (10%)	54	51
Construction Contingency (20%)	119	113
Subtotal	**712**	**676**
<u>Approach Bridge</u>		
Approach Construction Subtotal	62	103
Design Contingency	9	16
Construction Contingency (20%)	14	24
Subtotal	**85**	**143**
Grand Total (Rounded)	**800**	**820**

11.4. Life Cycle Cost

In the previous section, a construction cost was presented that represents the estimated construction, or initial, cost in 2008 dollars. Given the different structure types being considered, it is appropriate to also consider the life-cycle costs involved for each alternative.

Life cycle costs represent the anticipated future expenditures to maintain the bridge through its service life, 120 years. The future expenditures include such items as routine inspection costs, replacement of bridge elements that wear out and need to be replaced within the design life (such as deck overlay riding surface, bearings and joints), items that have a service life less than the overall design life and therefore must be replaced (such as lights, tower elevators, inspection gantries), and allowances for normal maintenance over time. These costs may be different for the various structural options and therefore a life cycle cost analysis is instructive to compare the alternatives on a future-needs basis.

It is common to present the future expenditures identified in the life cycle cost as the present worth values of the future expenditure, brought back to 2008 dollars using standard economic principles and a "Discount Rate" value. The discount rate represents a combination of inflation and interest rate (time value of money). The procedures of the Life Cycle Cost Analysis (LCCA) in this evaluation follow FHWA recommendations and those presented in NCHRP Report 483 - Bridge Life Cycle Cost Analysis.

The life cycle cost analysis is evaluated with a range of discount rate values of 3%, 5% and 7% to demonstrate the sensitivity of the analysis. Current recommendations from the U.S. Office of Management and Budget are to use a 3% real discount rate.

A summary of the initial construction cost and the LCCA costs are shown in **Table 15** below. These costs are presented in present value, i.e., 2008 dollars.

Table 15. *Life Cycle Cost Estimates (in $millions)*

Crossing:	**X-10(B)**	
Option:	**4**	**7**
Discount Rate	**Cable-Stayed**	**Suspension**
3%	472	514
5%	456	500
7%	450	495

Note: Life Cycle Costs are for Main Bridge only and reflect the length of the various main bridge options as shown in Appendix A Drawings, and are not adjusted to a common length.

The detailed life cycle cost evaluations are included for each bridge option in **Appendix C**.

11.5. Risks and Risk Assessment

The cost estimates presented above include a design contingency that recognizes the current level of design development. For the main bridge and the U.S. approach this contingency is 10% of estimated construction cost. For the Canadian approach structure this contingency is 20%. As the design approaches 100%, this contingency will be reduced to zero.

The above cost estimates also include a 20% construction cost contingency that reflects a judgment of the possible variation in construction bid costs within the construction industry. This contingency reflects normal variations in construction costs due to the competitive aspects of the construction marketplace. Some level of construction cost contingency will need to be carried forward on all estimates, however as the design is completed the value may be reduced.

In addition to the normal construction cost contingency noted above it should be noted that there are sometimes additional factors that may influence construction costs that are outside of normal construction variations. Examples of these types of factors include:

- Adjustment of material costs in response to global market factors, such as structural steel price adjustments in recent years due to high foreign demand for steel.
- Price control impacts on materials, such as the impact of a "Buy American" clause for structural steel. The pricing in the above estimates include the cost premium of $1.30 per Kg of structural steel, assuming "Buy America" applies to the structural steel. This results in a premium cost ranging from $14 million to $16 million, based on the estimated steel quantities for the various bridge alternatives. A corresponding cost savings may therefore be realized if this requirement is removed.

12. Considerations for Subsequent Development

As the structures are carried forward into Preliminary Design additional study should be carried out including the following areas.

12.1. New Materials

There are new materials and coatings which should be considered for the final design. Among these are:

- High Performance Concrete (HPC)
- Self-Consolidating Concrete (SCC)
- High Performance Steel (HPS)
- Advanced protective rebar coatings
- Alternative rebar materials (FRP and stainless steel clad)
- Alternative paint systems.

12.2. Aerodynamic Stability Investigations

Cable-stayed and suspension bridges are subject to dynamic response under wind loadings. The bridge concept evaluations to date have not performed any project specific aerodynamic evaluations, and have based the proposed designs on engineering judgment based on performance of other similar designs. Further development of the bridge concept should include project specific wind studies including the following:

- Site specific wind evaluation to establish wind speeds
- Evaluation of static drag for the proposed bridge deck
- Evaluation of static drag for the proposed towers

Though not necessary during the early preliminary design phase, for completeness the following studies are needed for final design:

- Wind stability evaluation (wind tunnel testing) of the proposed bridge deck in its completed condition including potential response to vortex shedding, buffeting, and flutter
- Wind stability evaluations of the proposed bridge deck at critical construction stages including potential response to vortex shedding, buffeting, and flutter
- Wind stability analysis of the completed free-standing tower
- Wind stability analysis of the tower in intermediate erection stages.

In the structure study a generic pedestrian railing is shown. The design of a specific railing and/or fencing should be examined in concert with the aerodynamic studies.

12.3. Inspection Access

In future stages of the project, it is recommended to consider a scoping exercise for consideration of maintenance and inspection access. Provisions for jacking of the superstructure at all locations that have bearings that will require future maintenance may be considered. All internal parts of the structure should be accessible for inspection. The interior of the box girder, towers/pylons, and anchorages should be provided with lighting and electrical outlets for use during inspections. Permanent moveable platforms may be considered for underbridge inspection and maintenance on spans where access by snooper or lift is either impractical or significantly affects the operation of the facility. Options for tower/pylon access should be developed.

12.4. Durability

In future stages of the project, specific design goals for durability, service life of specific elements and appropriate maintenance schedules should be developed. This may include:

- Durability of concrete elements developed by a corrosion analysis that includes factors such as mix design, specific admixtures, concrete covers, rebar type and anticipated maintenance.
- Protective coating recommendations for structural steel including anticipated maintenance
- Cable protection strategies (suspension or stay cable) and anticipated maintenance
- Deck overlay systems and strategies.

12.5. Structural Monitoring

Structural monitoring systems are a rapidly advancing technology that can provide owners with long term performance data of the structure to guide maintenance operations, or real-time performance evaluations that can provide safety assurances and incident management capabilities. As part of the ongoing development of the bridge concept, an overall strategy for monitoring systems can be developed. The specific technologies are probably better specified later in the development process to take advantage of the latest in state-of-the-art developments in communications and monitoring equipment.

12.6. Security/Hardening

Today's major bridge designs consider not only design for natural hazards, but also consider design and protection strategies for intentional acts to disrupt the performance of the structure. This is particularly important for a high-profile project such as the Detroit River International Crossing. Some of the factors that should be considered as the project develops include:

- Development of secure procedures for document control of the developing design documents, with the goal of limiting access to sensitive design information and reports.
- Development of specific goals for the structural design development, including redundancy requirements, hardening requirements, stand-off distances, and means of limiting access.
- Development of specific hazard loading and performance of specific hazard analysis for design.
- Development of secure access provisions while meeting the needs of inspection and maintenance.
- Development of any monitoring strategies.
- Development of incident management strategies.

12.7. Aesthetics and Context Sensitive Solutions (CSS)

The bridge concepts presented in this report were developed with the primary goals of :

- Development and confirmation of the viability of the structural concept
- Development of probable construction cost for the concept
- Determination of their environmental impacts

The Detroit River International Crossing Bridge represents a major structure and warrants consideration of the visual attributes and quality of the crossing. While the aesthetic development has not been a primary objective of the conceptual development, there has been an awareness of the magnitude and importance of the crossing and attention was given to providing a logical and well proportioned structure.

Subsequent development of the design(s) should specifically address the visual quality and focus on the aesthetic development of the design. A series of Context Sensitive Design Workshops were conducted in parallel with the development of the bridge concepts and the results of those workshops should be factored into the subsequent visual development of the bridge(s). **Section 3** of the Engineering Report details the Context Sensitive Solutions process that included considerations of the bridge form and aesthetics.

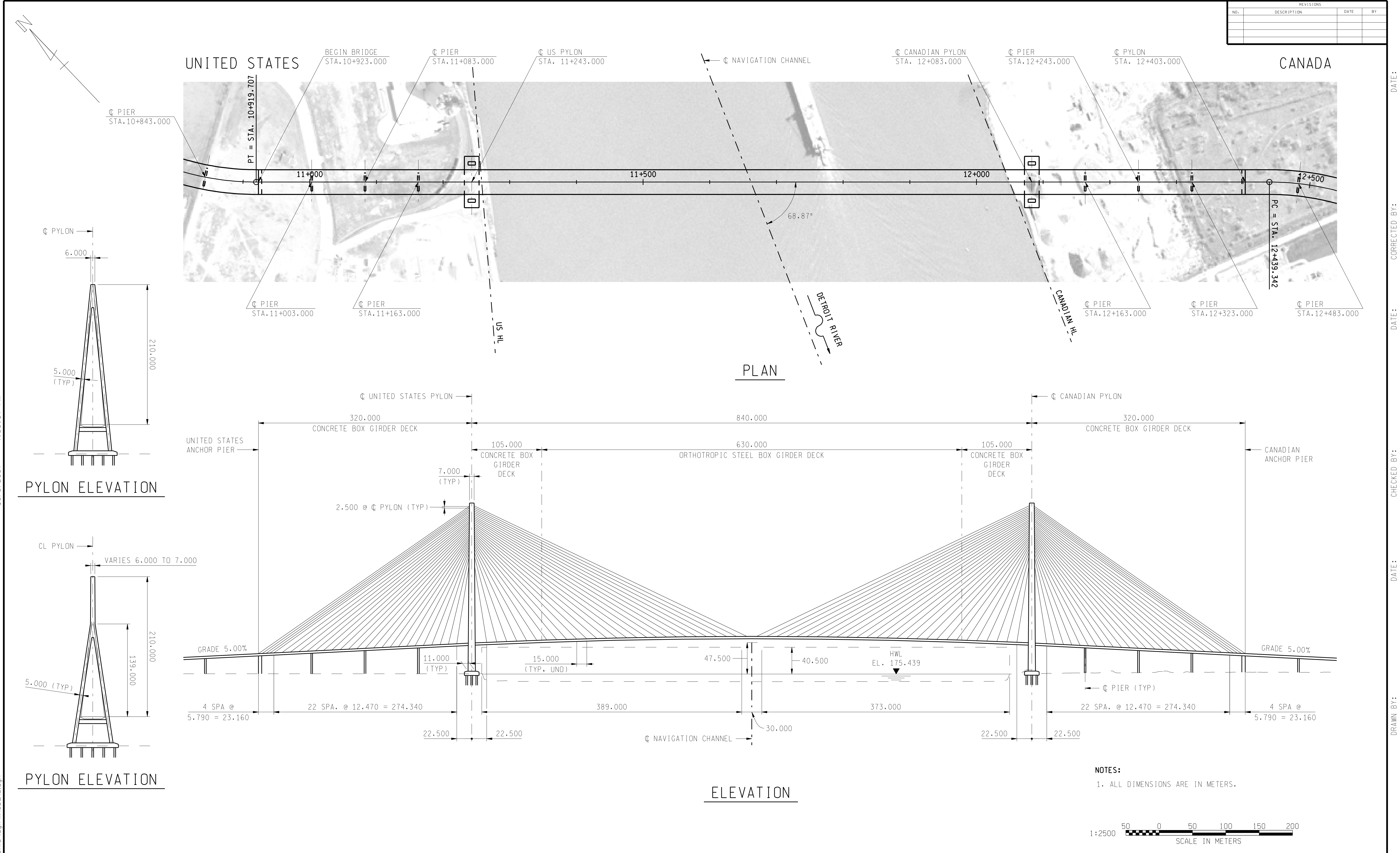
12.8. U.S. Approach Bridge

For the U.S. approach bridge several potential refinements can be investigated during the final design stage of the project:

- Investigate providing transverse expansion capability for the deck and eliminate the longitudinal deck joint where feasible. In addition to reducing maintenance, it would likely allow fewer girders to be utilized.
- Review the use of voided columns for the tall piers.
- Consider optimization of the 4-span continuous structural steel units by shortening end spans and lengthening interior spans in lieu of using constant span lengths.
- Consider using structural steel girders combined with the pre-cast concrete Michigan 1800 girders for the suspension span type main bridge. The structural steel girders would replace the post-tensioned, modified Michigan 1800 girders in the spans between the anchor block and main span tower. This decision would depend on cost at the time of construction.
- Consider providing enough width on the Canada bound side at initial construction to eliminate the need for a future widening if an eight lane section becomes justified.

- Pedestrian Railing: As noted above the design of the pedestrian railing will be part of the overall aerodynamic considerations for the main span structure. The approach pedestrian railing is shown as an MDOT 4-tube railing in this study to be conservative, however, a crash tested pedestrian railing is not necessary. The approach pedestrian railing should match the main span structure railing for aesthetic continuity. In addition, further consultation will be needed with US Customs and Border Protection with regard to the need or requirement for pedestrian fencing as the structure reaches lower elevations.

APPENDIX A – Main Bridge Plans



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DETROIT RIVER INTERNATIONAL CROSSING

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GENERAL PLAN & ELEVATION
CROSSING X10(B) OPTION 4

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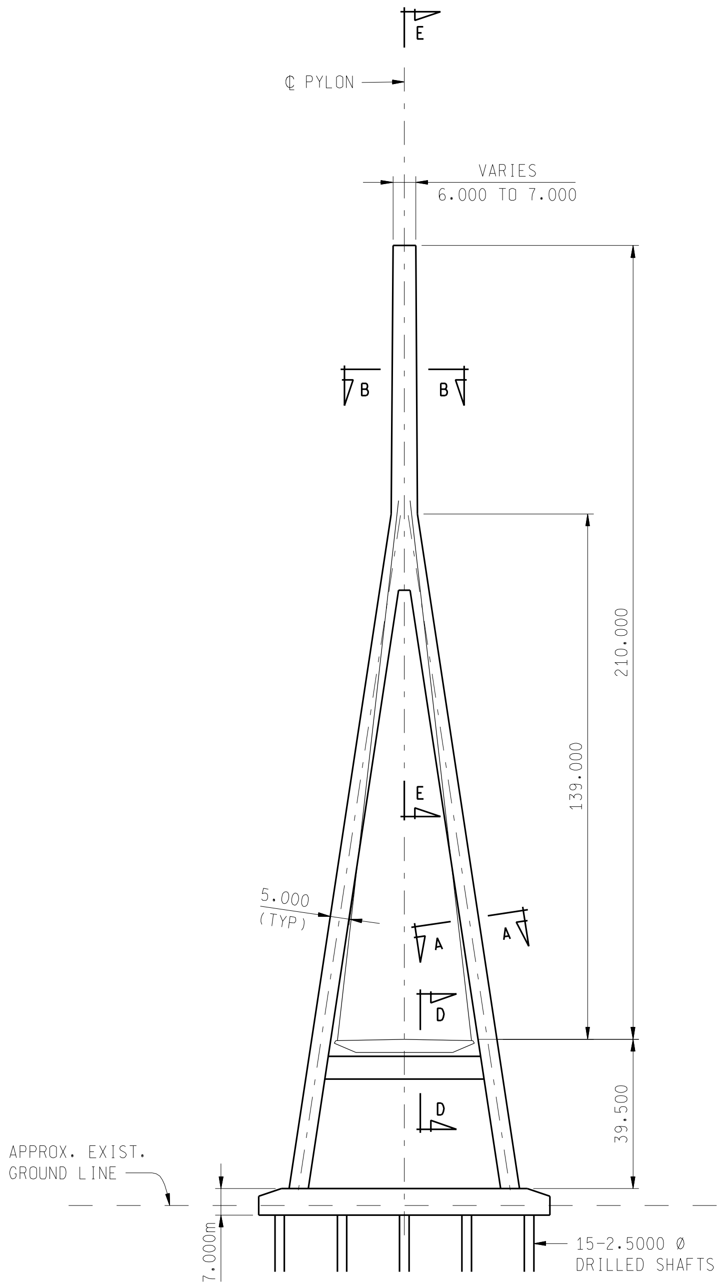

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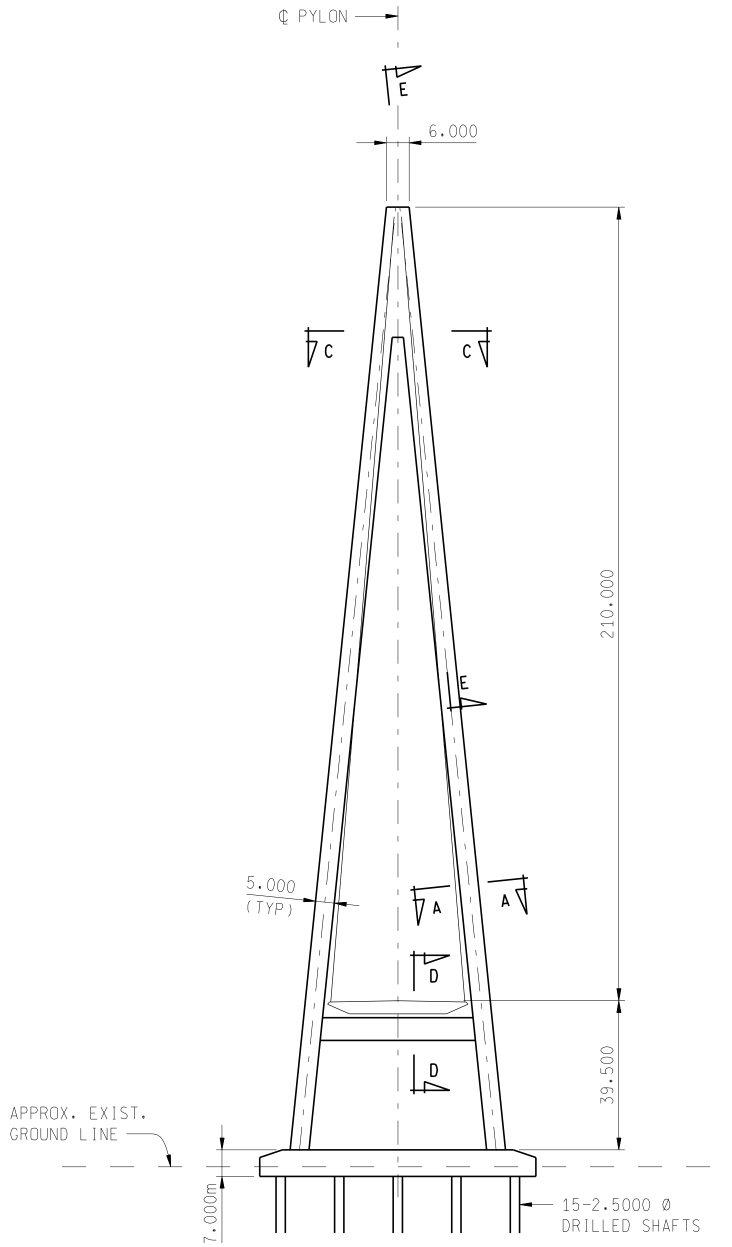

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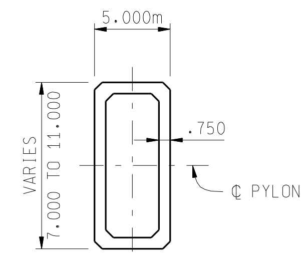
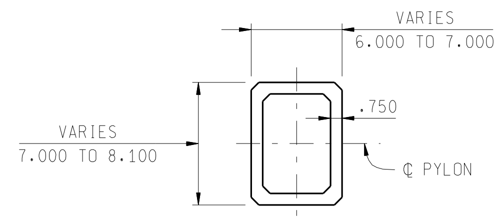


SECTION A-A



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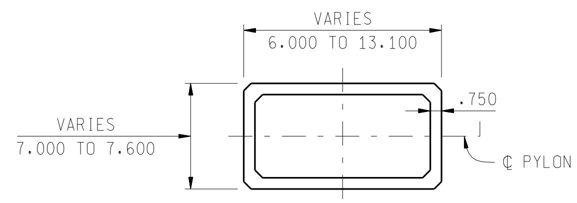

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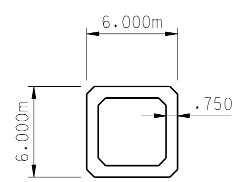

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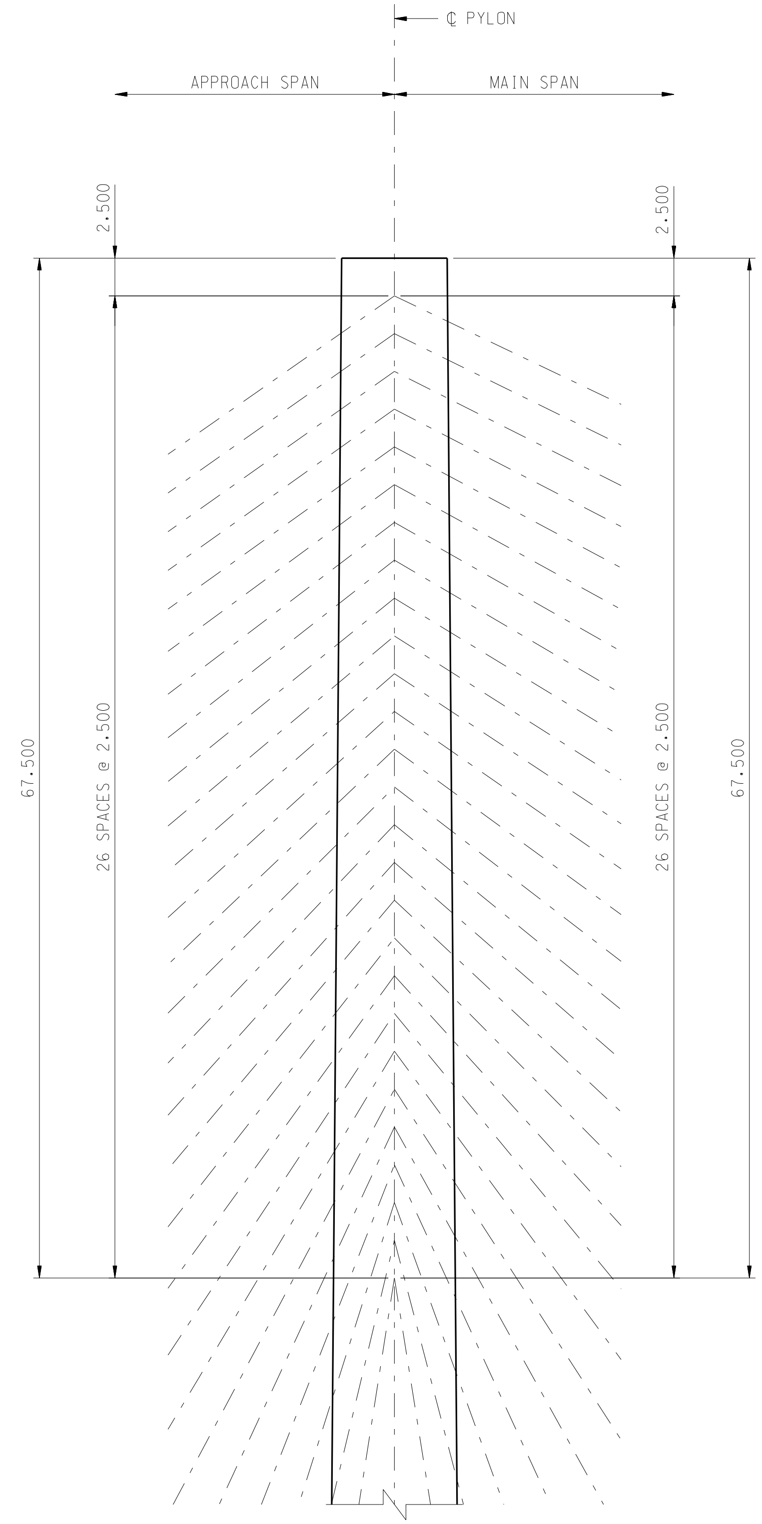

SECTION E-E

NOTES:

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DETROIT RIVER INTERNATIONAL CROSSING

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TOWER ELEVATIONS
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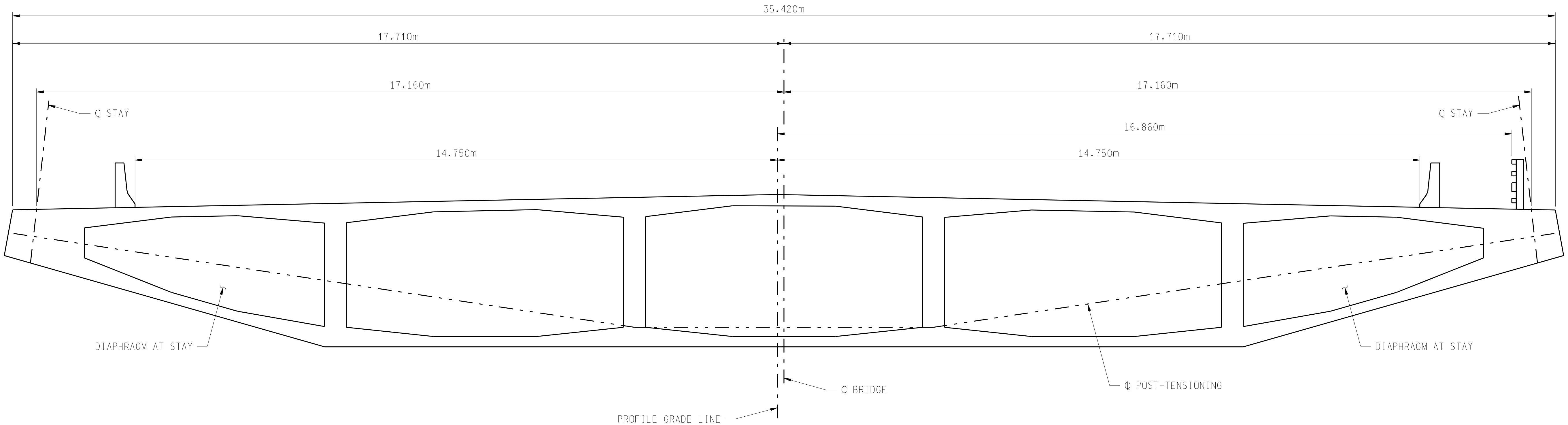

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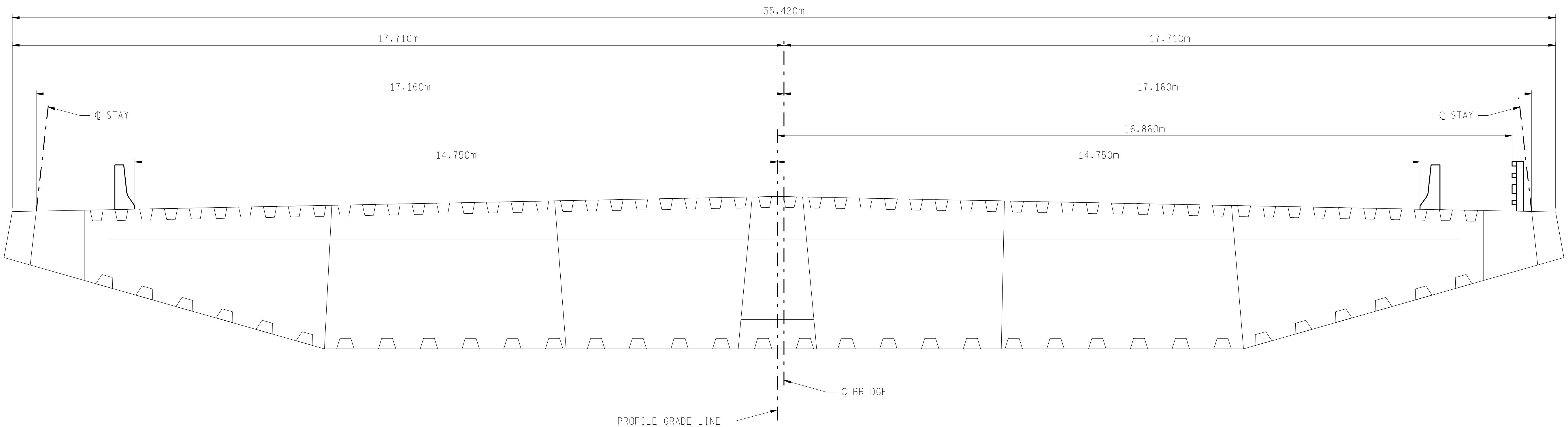

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DETROIT RIVER INTERNATIONAL CROSSING

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CROSS SECTION
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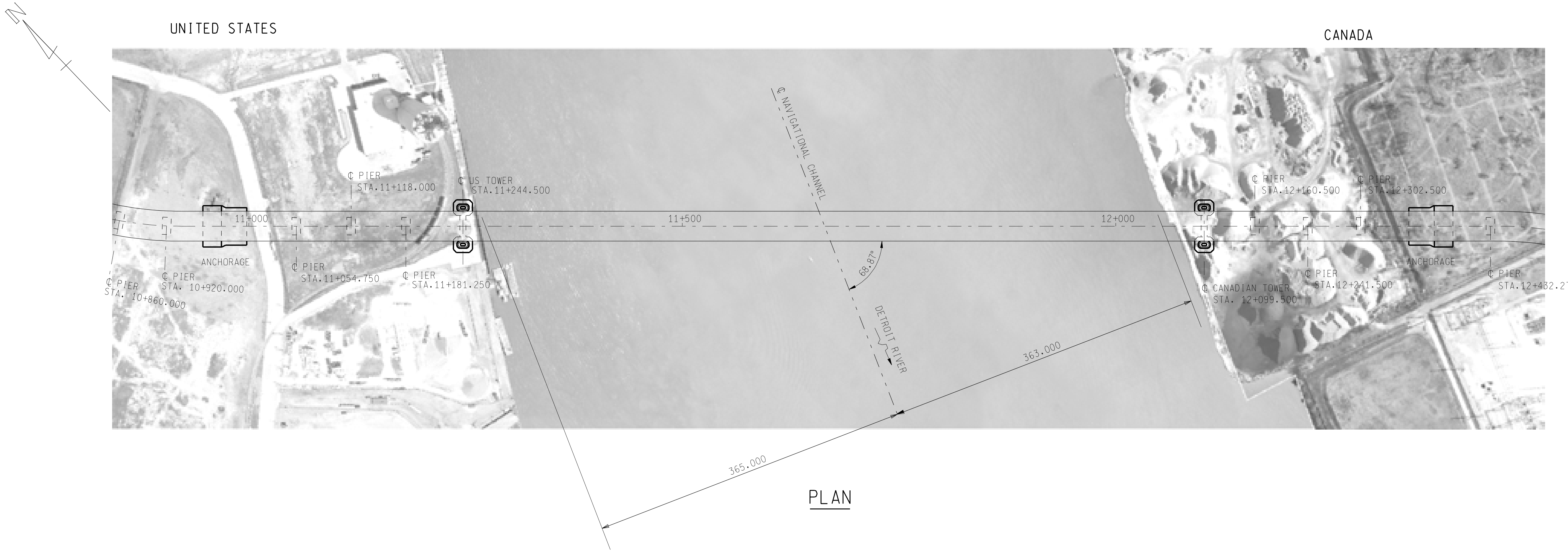

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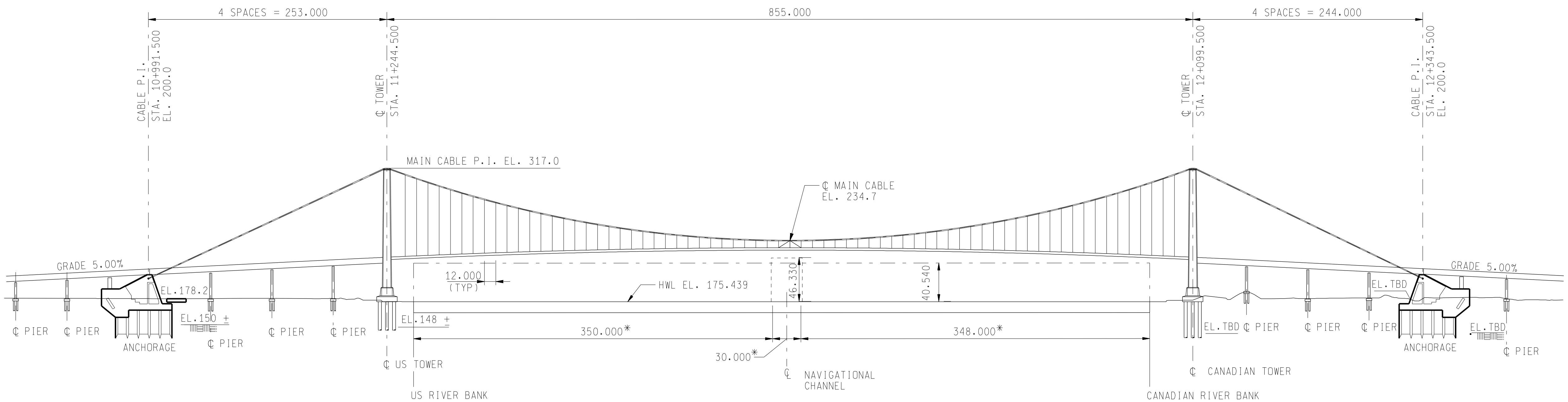

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ELEVATION

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DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

GENERAL PLAN & ELEVATION
CROSSING X10 (B) OPTION 7

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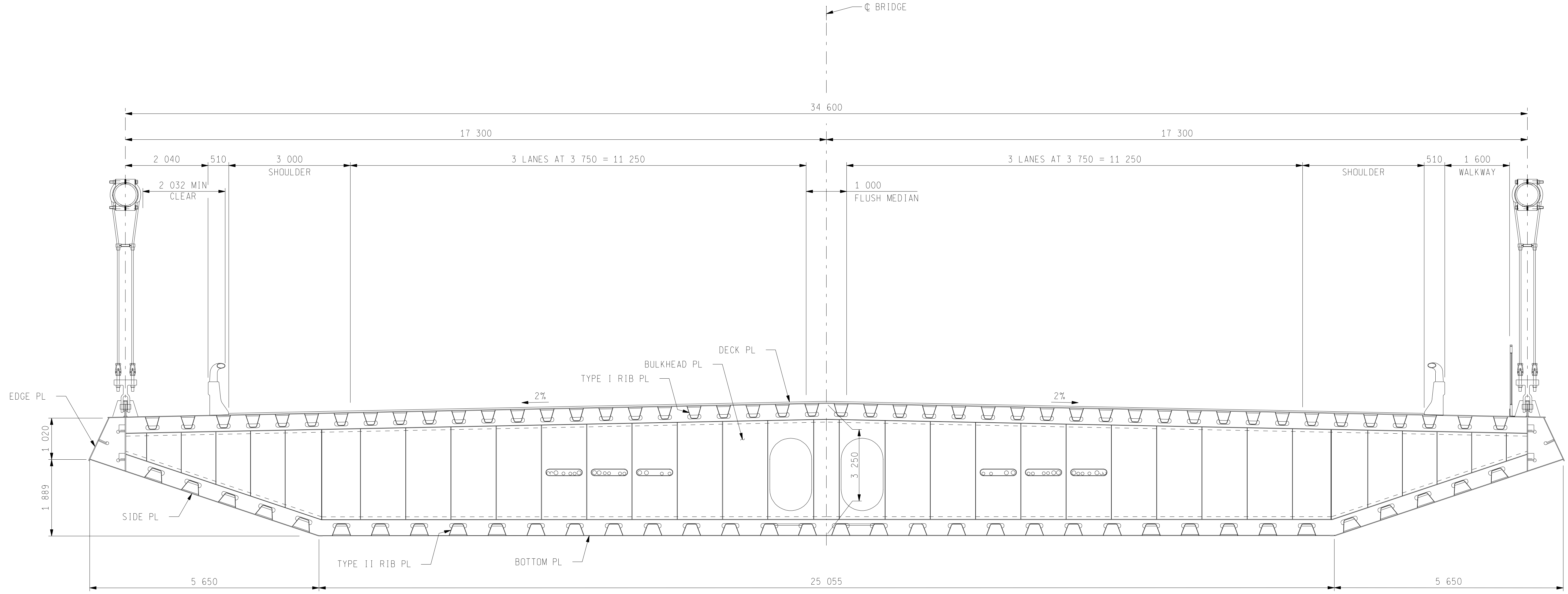

TYPICAL CROSS SECTION

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DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

BOX GIRDER SECTIONS
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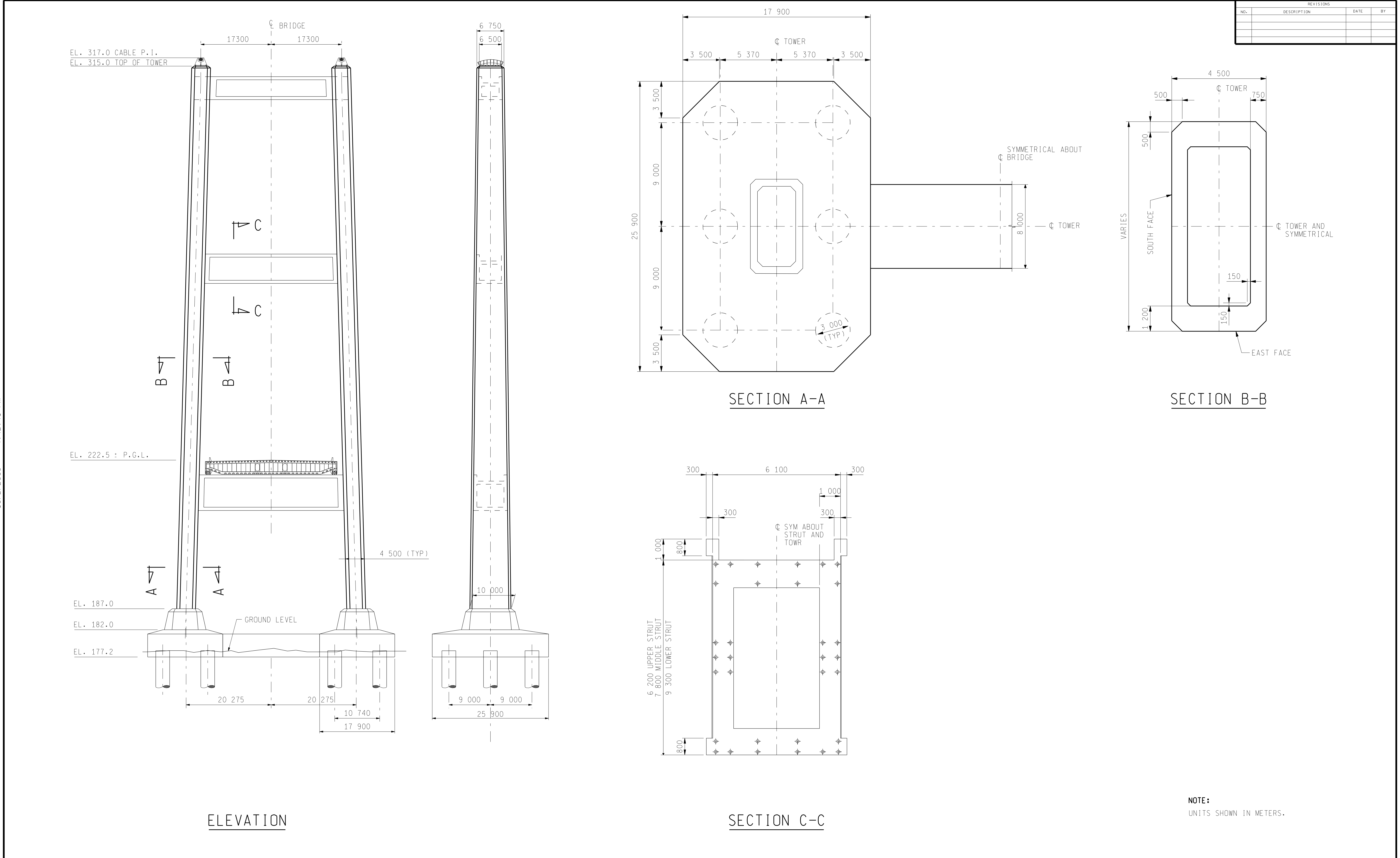

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DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

TOWER
CROSSING X10 (B) OPTION 7

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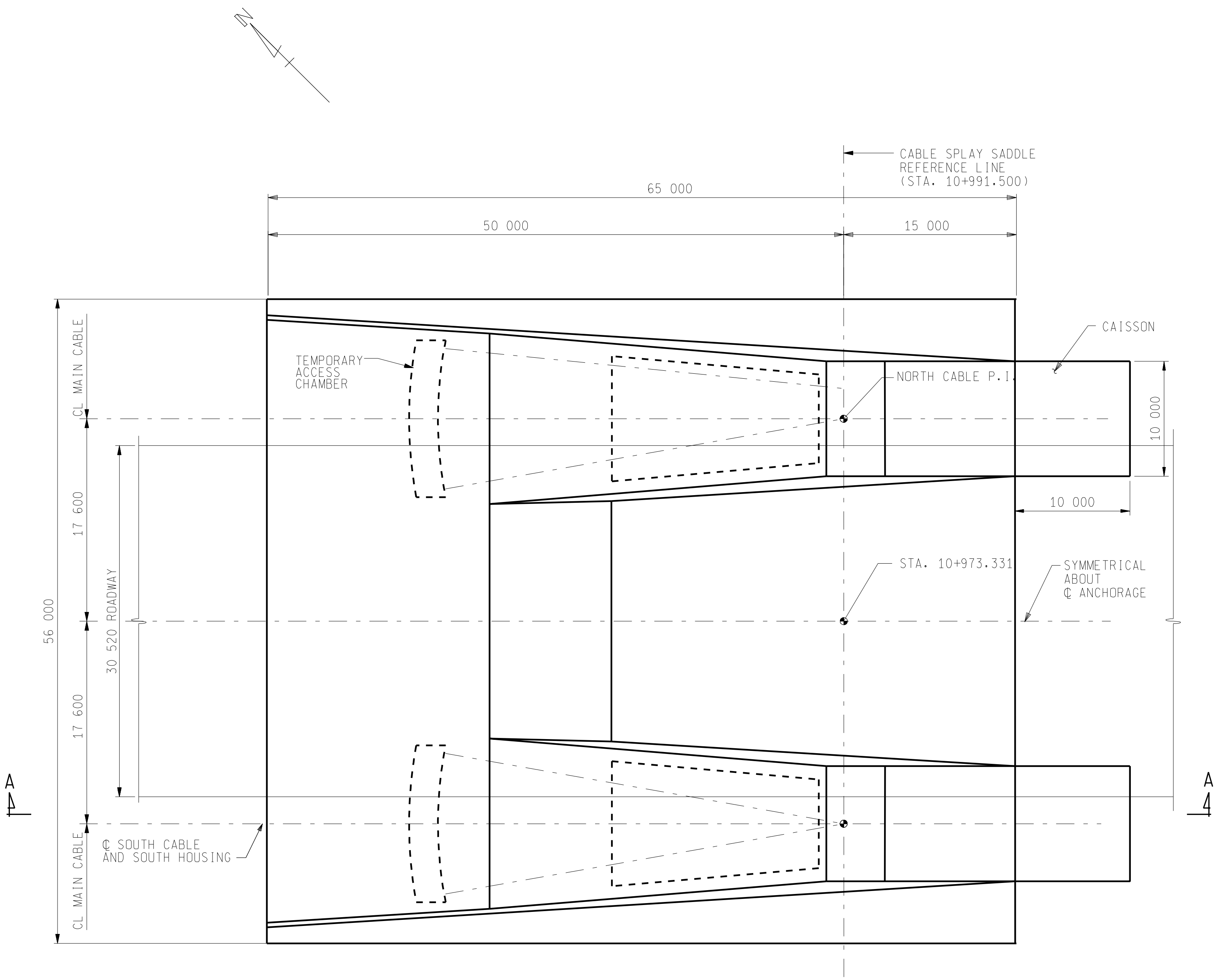

PLAN

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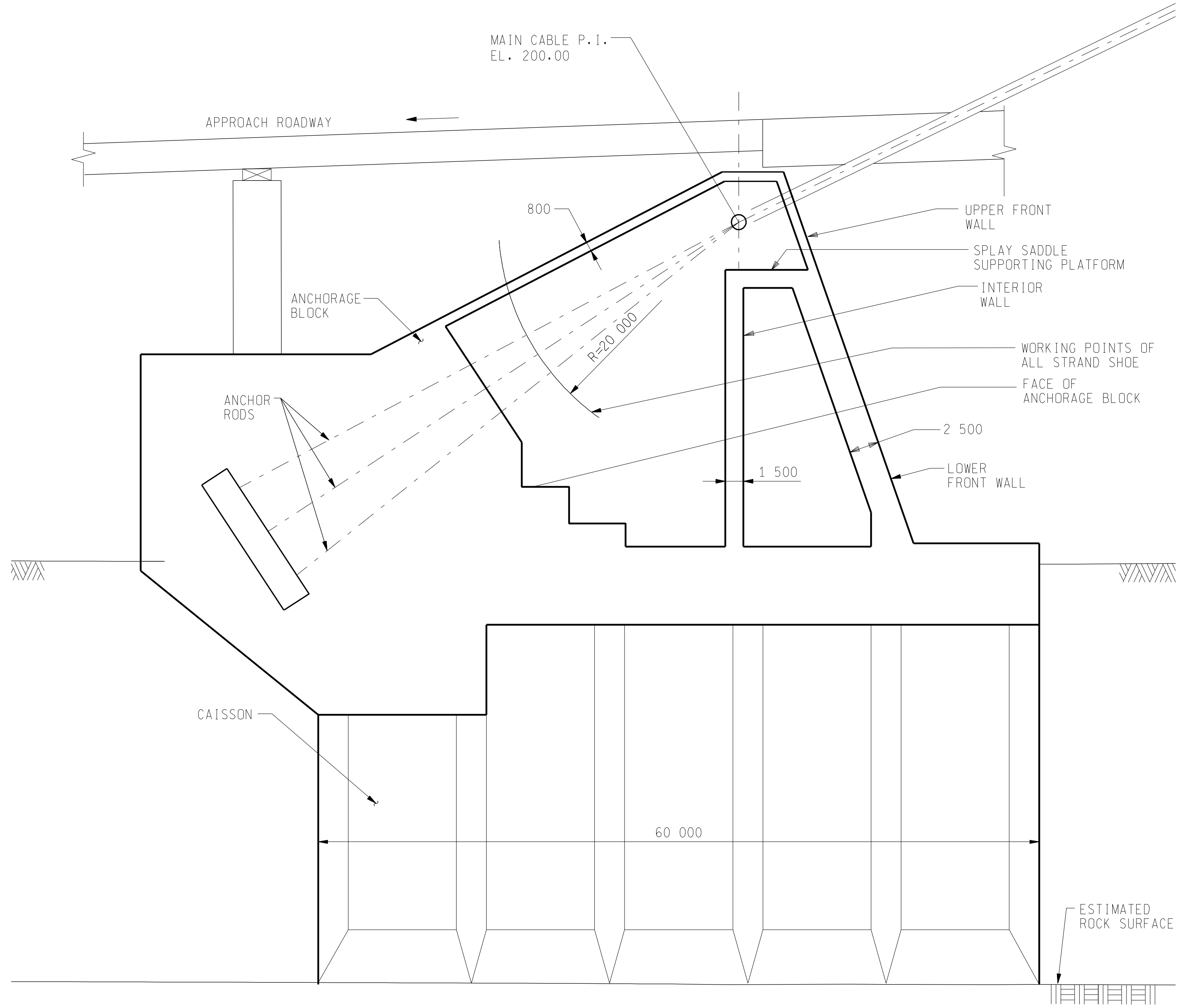

SECTION A-A

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DETROIT RIVER INTERNATIONAL CROSSING

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ANCHORAGE
CROSSING X10 (B) OPTION 7

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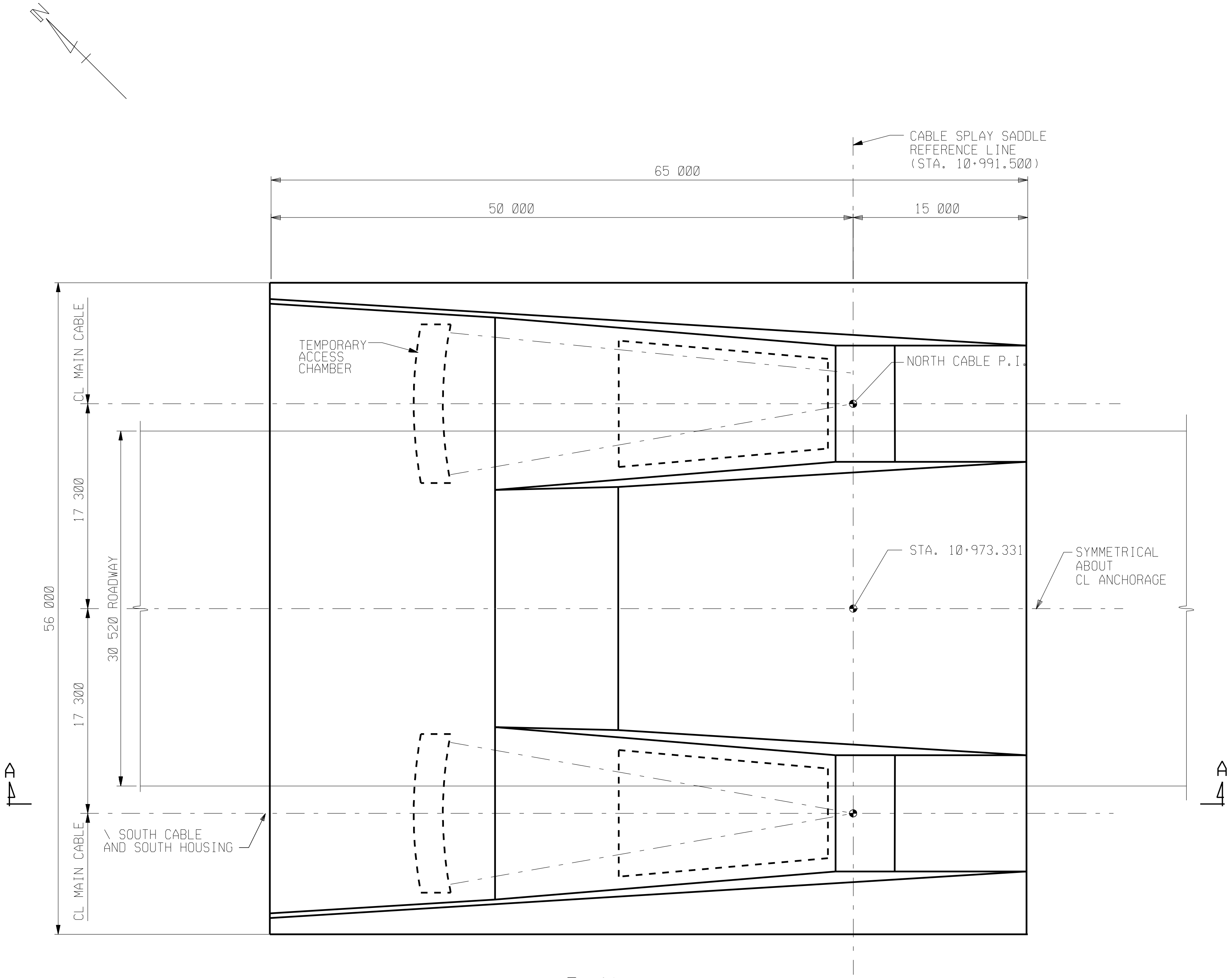

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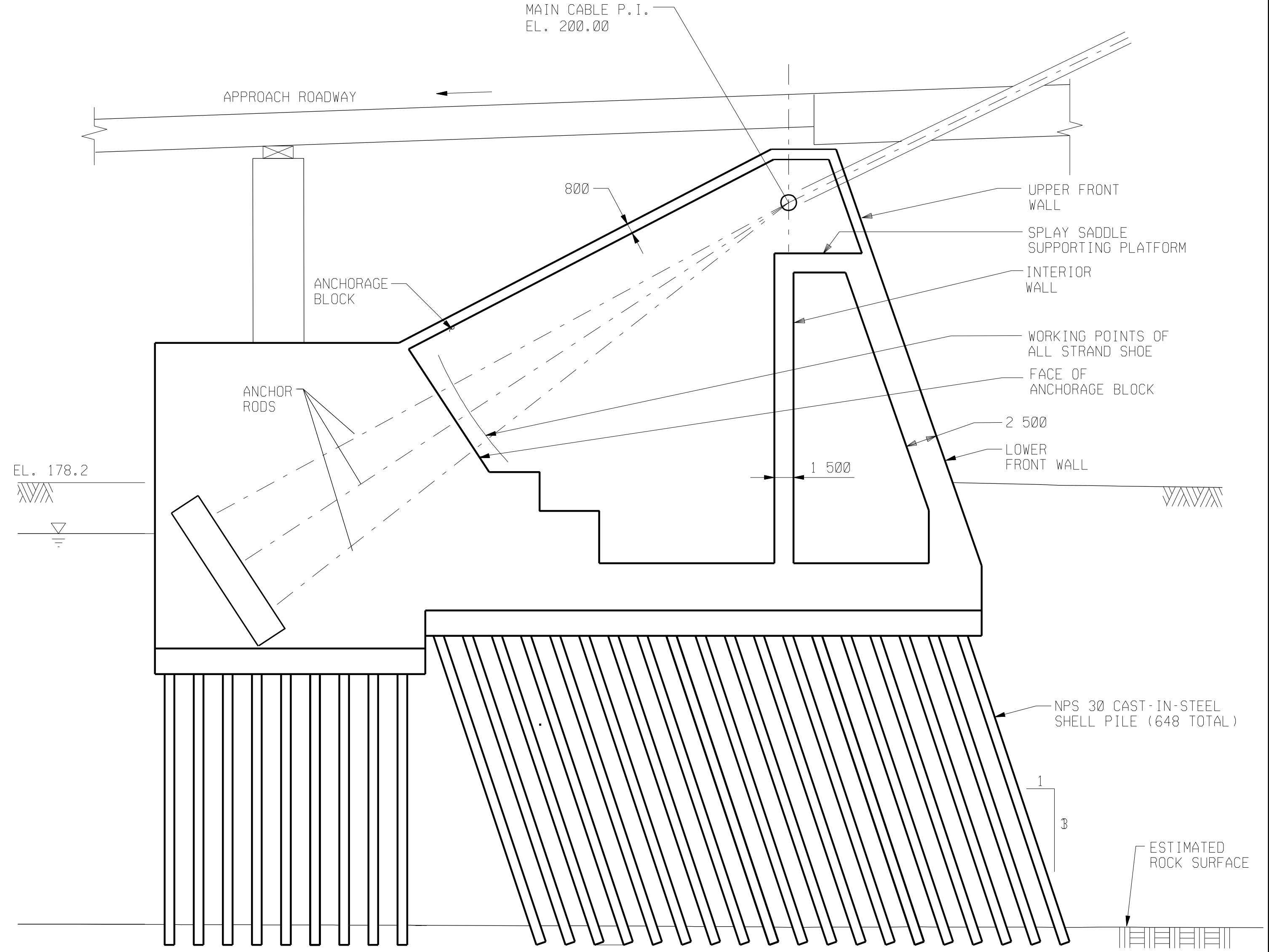

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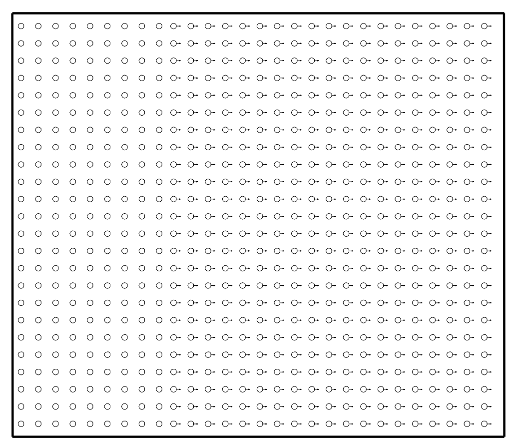
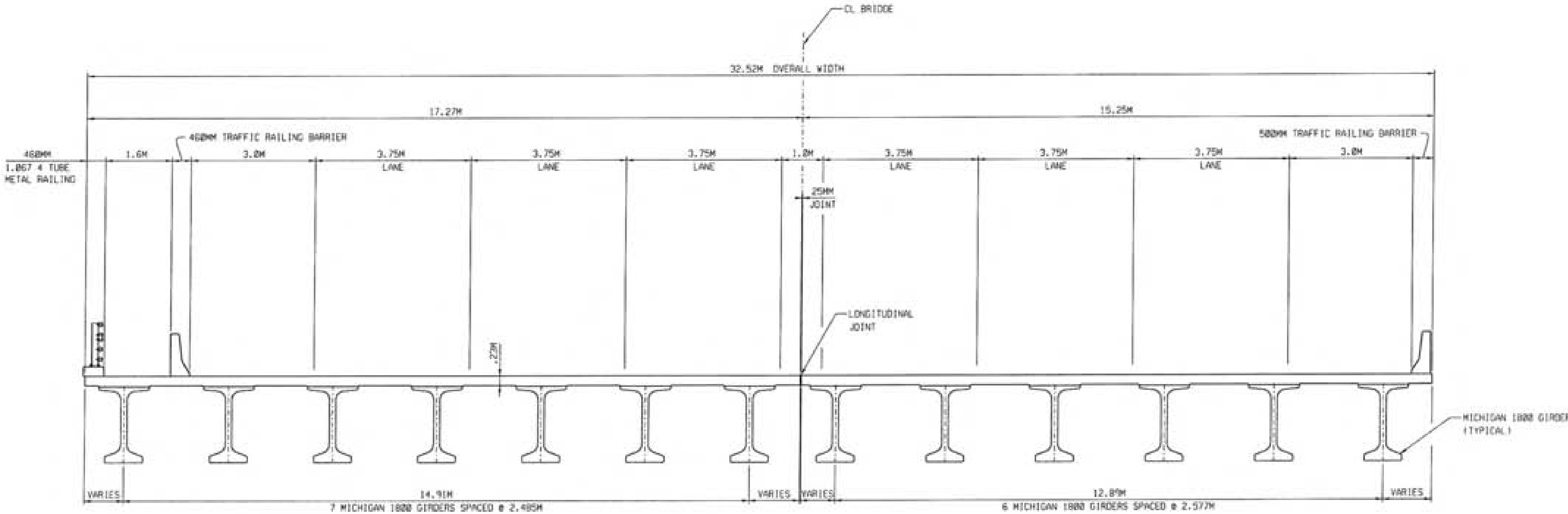


PILE LAYOUT PLAN

APPENDIX A – Approach Bridge Plans



TYPICAL SECTION

PRESTRESSED CONCRETE GIRDERS ALTERNATE 'A'

FIGURE 3-1A

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DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

TYPICAL SECTION ALTERNATE 'A'

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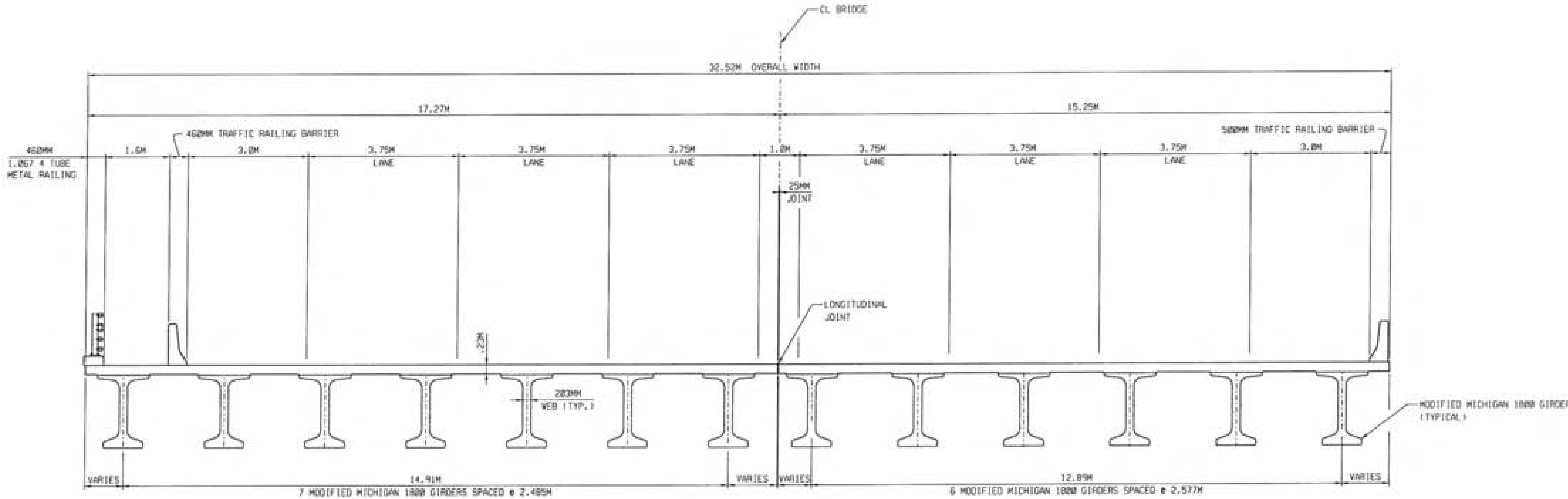


TYPICAL SECTION
PRESTRESSED CONCRETE GIRDERS ALTERNATE "A1"

(APPLIES ONLY TO SUSPENSION BRIDGE MAIN SPAN TYPE)
(GIRDERS SHOWN ARE POST-TENSIONED)

FIGURE 3-1B

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

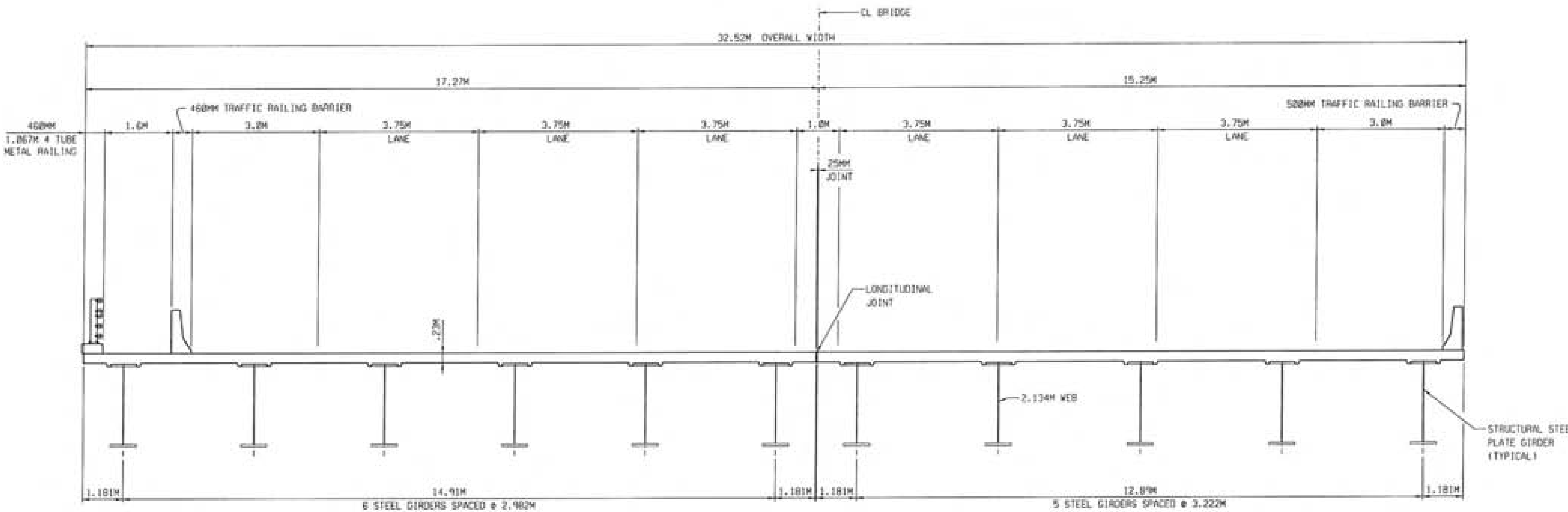
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

TYPICAL SECTION ALTERNATE "A1"

SCALE 1:100
DATE 10/2/2008

P:\646294\BRIDGE\CADD\REVISED\Typical_Section_ConcreteGirder_MODIFIED.dgn 10/2/2008 7:32:42 PM



TYPICAL SECTION

STRUCTURAL STEEL PLATE GIRDERS ALTERNATE "B"

FIGURE 3-2

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

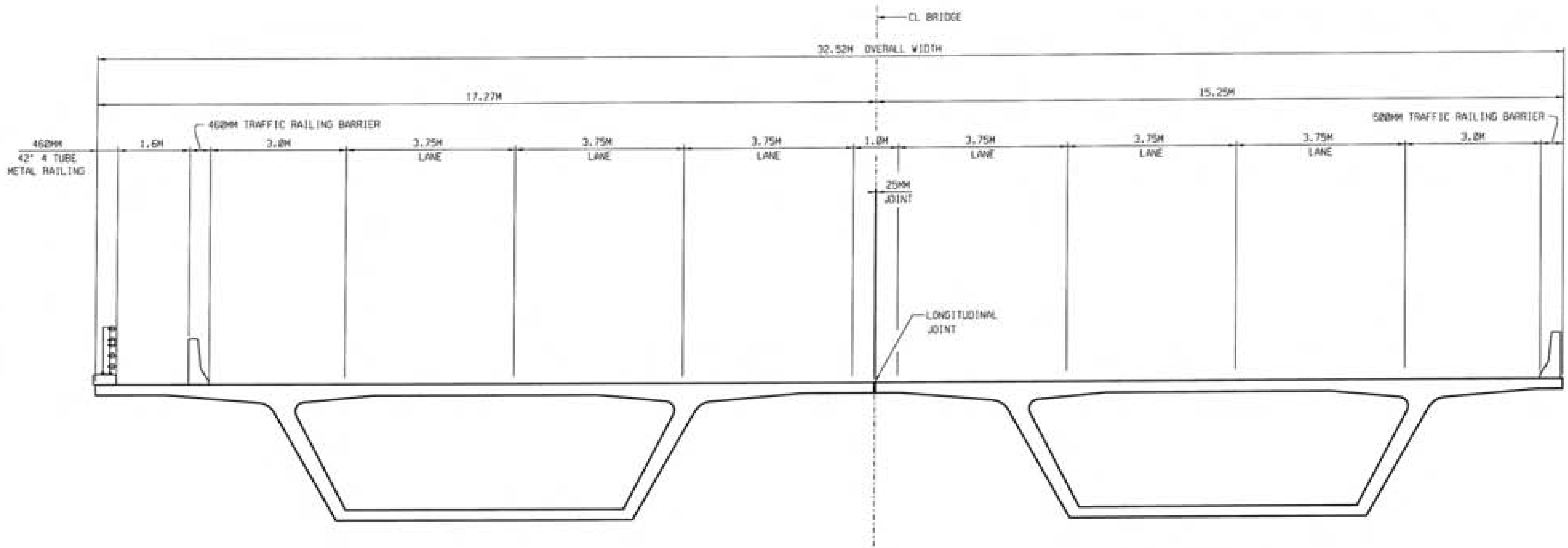
PARSONS

TYPICAL SECTION
ALTERNATE "B"

SCALE 1:100

DATE 10/2/2008

10/2/2008 7:27:09 PM

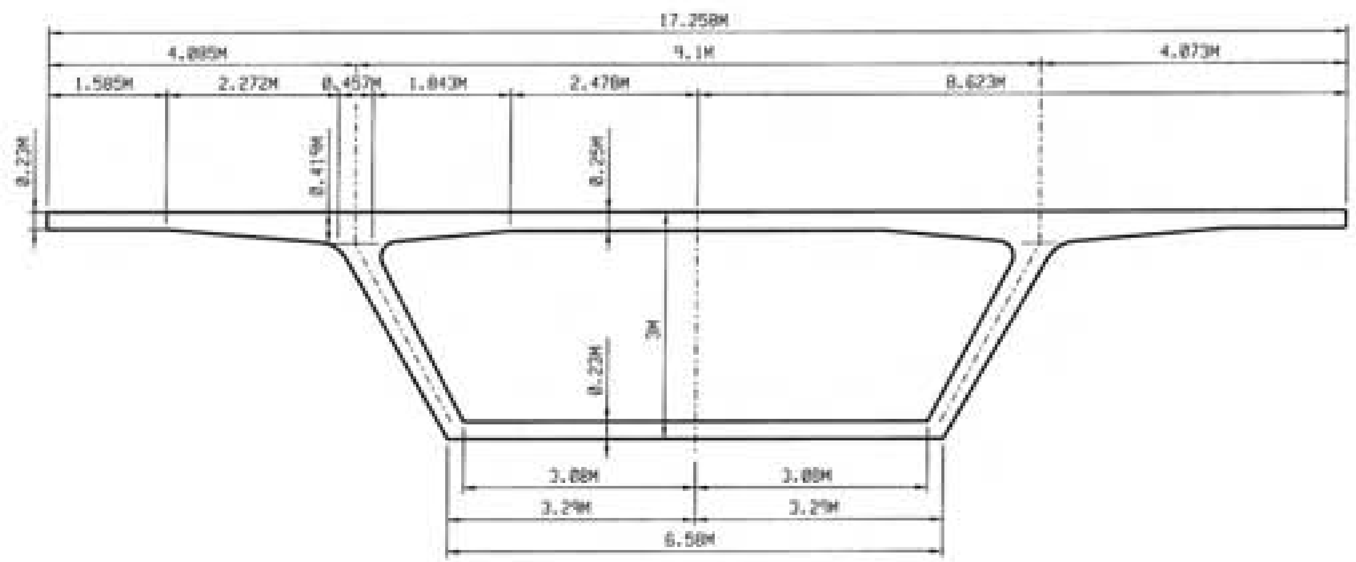


TYPICAL SECTION

(PRE-CAST SEGMENTAL- ALTERNATE "C")

FIGURE 3-3

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario	DETROIT RIVER INTERNATIONAL CROSSING	PARSONS	TYPICAL SECTION ALTERNATE "C"	SCALE 1:100 DATE 10/2/2008



SURFACE AREA: 8.54316 SQUARE M

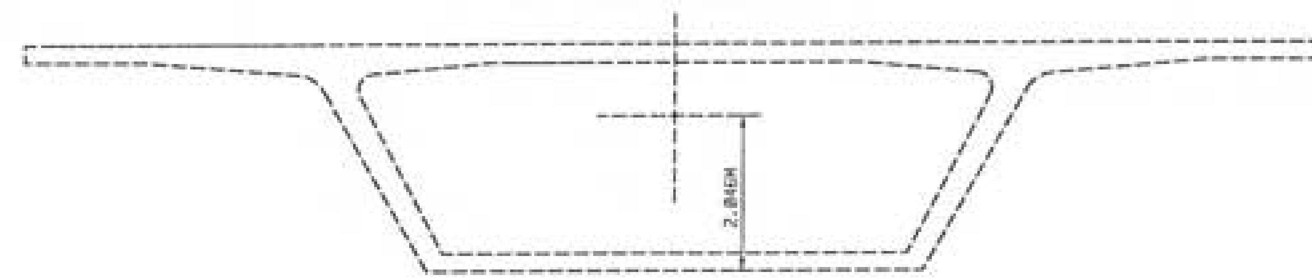
MASS PER AREA: 1
MASS: 8.54316

CENTROID: (1.48987E-009, 5.48536E-009, 0)

MOMENTS OF INERTIA: (10.5667, 157.165, 167.732)
PRODUCTS OF INERTIA IXY: -0.026097 IXZ: 0, IYZ 0

PRINCIPAL MOMENTS: 10.5667 (1, -0.000178016, 0)
PRINCIPAL MOMENTS: 157.165 (0.000178016, 1, 0)
PRINCIPAL MOMENTS: 167.732 (0, 0, 1)

RADII OF GYRATION: (1.11214, 4.28913, 4.43097)



TYPICAL SEGMENT

(PRE-CAST DETAILS)

FIGURE 3-4

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

TYPICAL SEGMENT
PRE-CAST SEGMENTAL

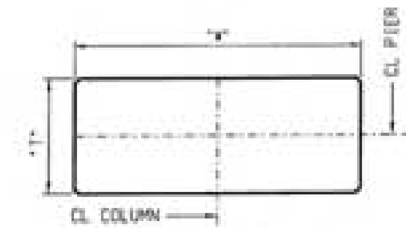
SCALE 1:100

DATE 10/2/2008

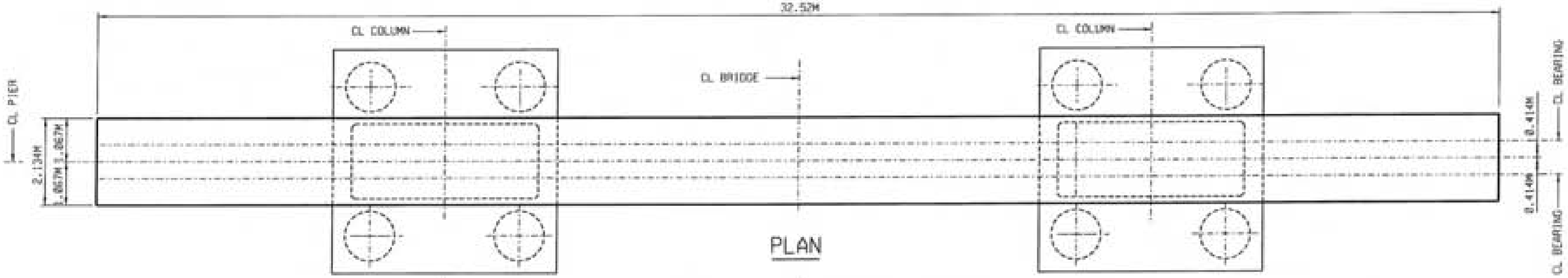
DIMENSION TABLE

BEAM TYPE	MAIN SPAN BRIDGE TYPE	COLUMN DIMENSIONS "W"	COLUMN DIMENSIONS "T"*	COLUMN DIMENSIONS "H"	FOOTING SIZE	DRILLED SHAFT SIZE	DRILLED SHAFT NUMBER
MICHIGAN 1800	CABLE STAYED	4.57M	1.83M	H < 18.29M	5.49 X 5.49 X 1.98	.91M	4
		4.57M	1.83M	H > 18.29M	5.49 X 5.49 X 1.98	1.22M	4
	SUSPENSION	4.57M	1.83M	H < 18.29M	5.49 X 5.49 X 1.98	.91M	4
		4.57M	1.83M	18.29M < H < 27.43M	5.49 X 5.49 X 1.98	1.22M	4
		4.57M	(TAPER) 2.74M TO 1.83M	H > 27.43M	7.62 X 7.32 X 2.59	1.22M	5
STEEL	CABLE STAYED	4.57M	1.83M	H < 18.29M	6.40 X 6.40 X 2.44	1.22M	4
		4.57M	(TAPER) 2.74M TO 1.83M	H > 18.29M	6.40 X 6.40 X 2.44	1.22M	4
	SUSPENSION	4.57M	1.83M	H < 27.43M (EXP. PIER)	6.40 X 6.40 X 2.44	1.22M	4
				H < 27.43M (FIXED PIER)	7.62 X 6.40 X 2.59	1.22M	5
		4.57M	(TAPER) 2.74M TO 1.83M	H > 27.43M	7.62 X 7.32 X 2.59	1.22M	5

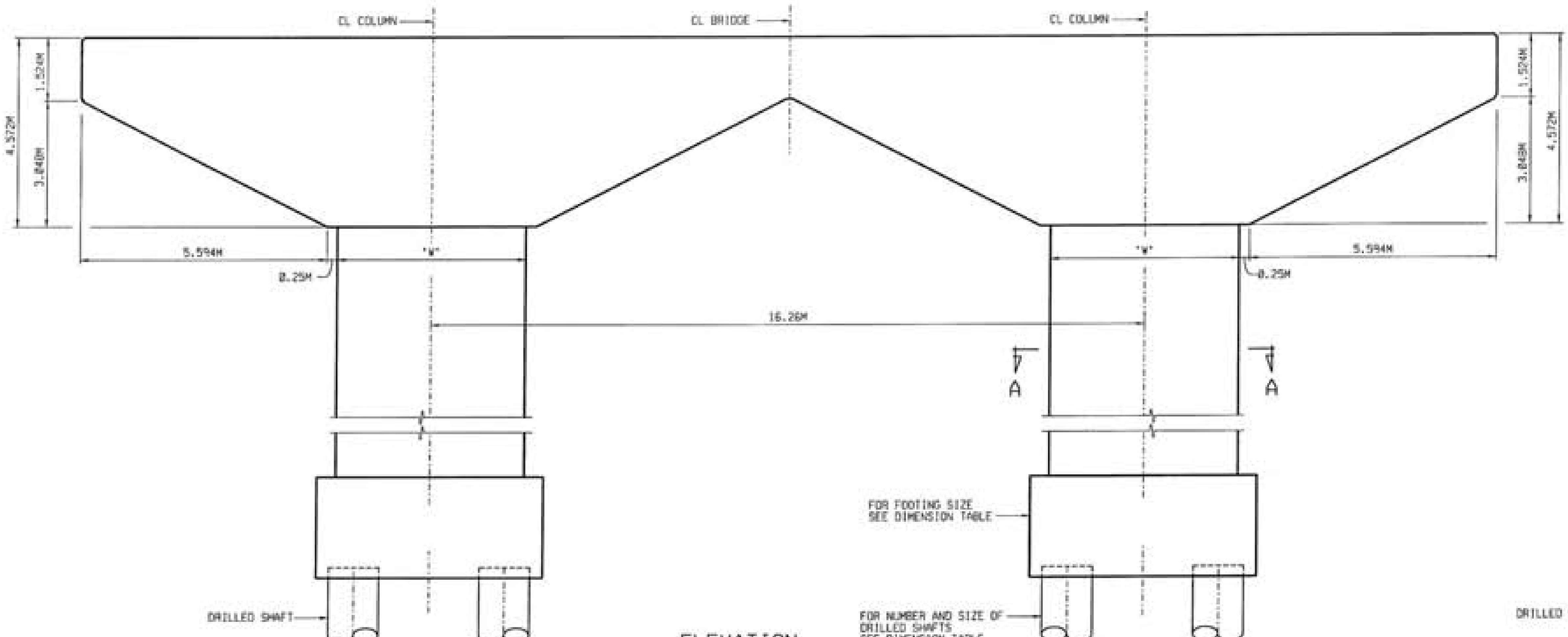
* - ALL THE TAPERED DIMENSIONS ARE FROM BOTTOM TO TOP.



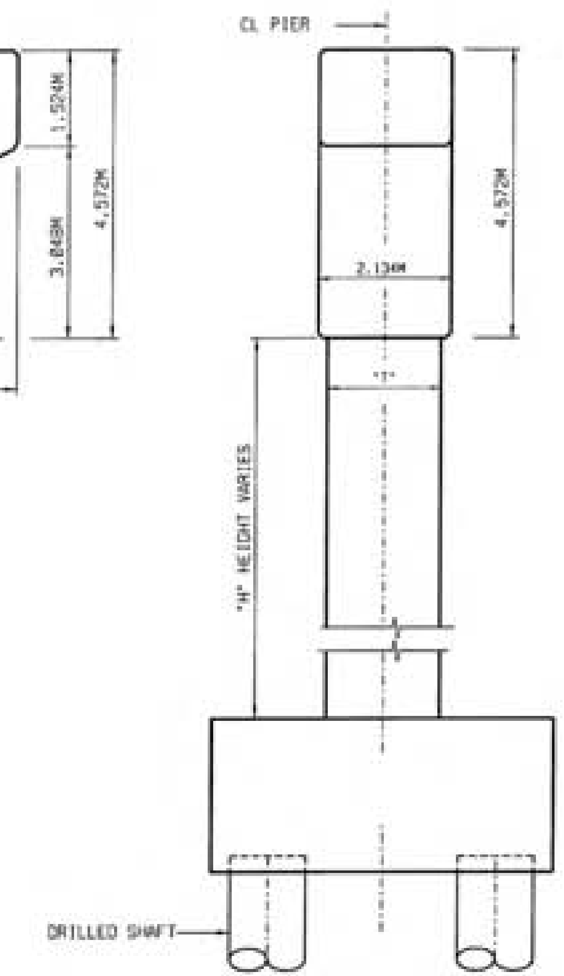
SECTION A-A



PLAN



ELEVATION



SIDE VIEW

FIGURE 3-5

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

PIER TYPE 1

SCALE 1:100

DATE 10/2/2008

P:\646294\BRDGE\CADD\REVISED\pier_01.dgn 10/2/2008 7:09:06 PM

DIMENSION TABLE					
BEAM TYPE	MAIN SPAN BRIDGE TYPE	COLUMN DIMENSIONS "H"	FOOTING SIZE	DRILLED SHAFT SIZE	DRILLED SHAFT NUMBER
SEGMENTAL	CABLE STAYED	ALL HEIGHTS	5.49 X 5.49 X 1.98	1.22M	4
	SUSPENSION	H < 27.43M	5.49 X 5.49 X 1.98	1.22M	4
		H > 27.43M	7.62 X 7.32 X 2.59	1.22M	5

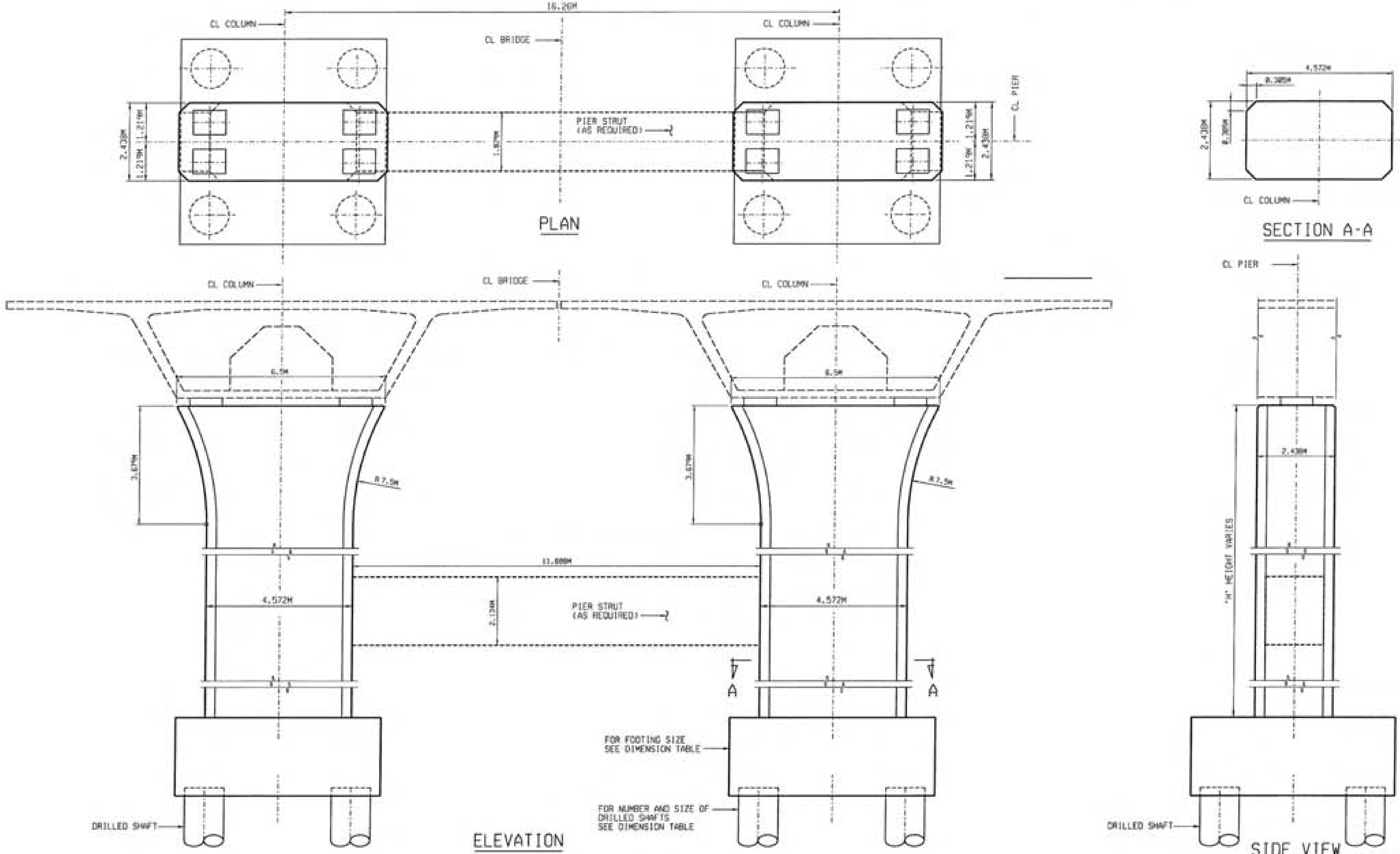


FIGURE 3-6

P:\545294\BRIDGE\CADD\REVISED\pier_02.dgn 10/2/2008 7:09:52 PM

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

PIER TYPE 2

SCALE 1:100

DATE 10/2/2008

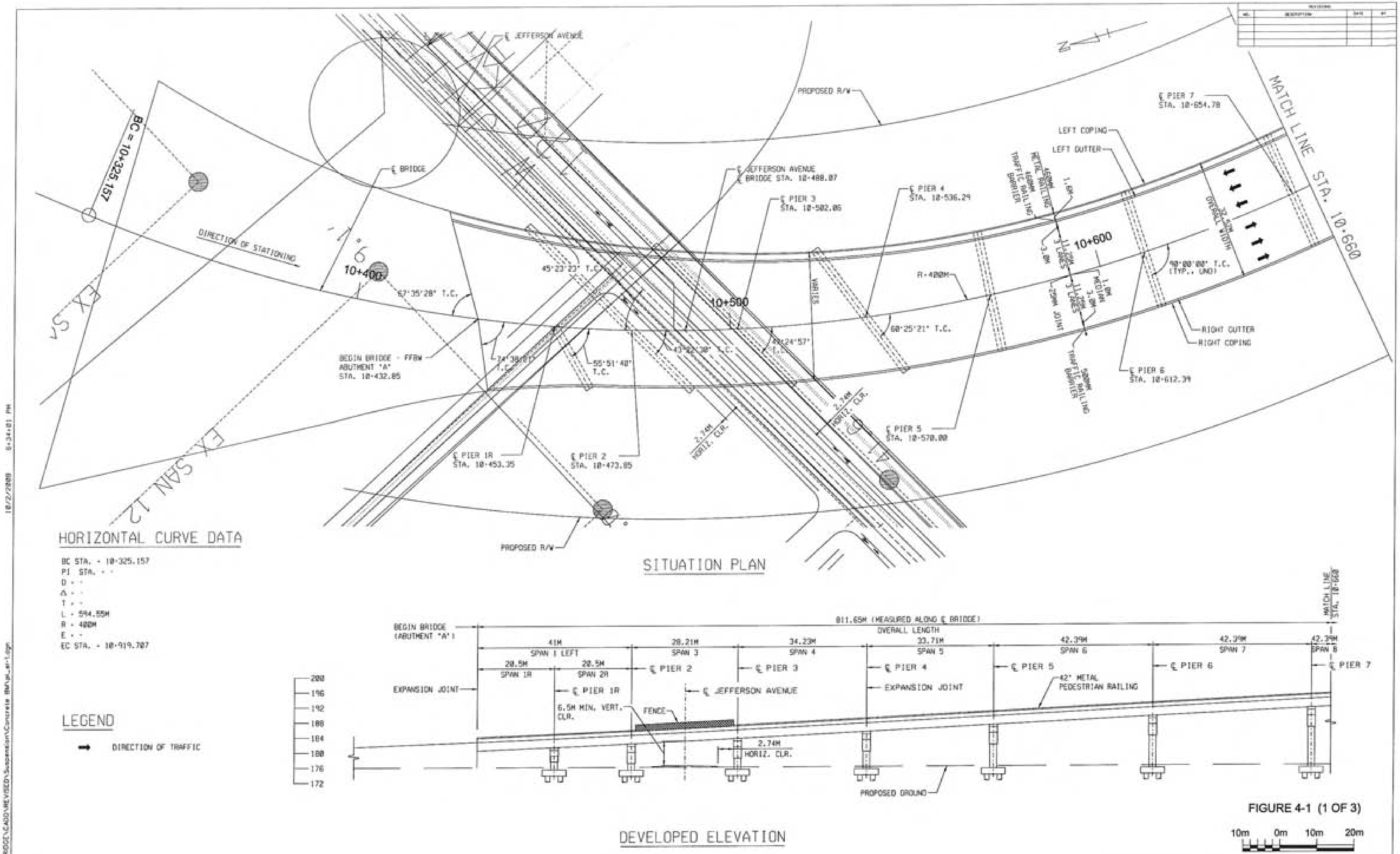


FIGURE 4-1 (1 OF 3)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE A1
PRESTRESSED CONC. BEAMS (SUSPENSION BR)

SCALE 1:100

DATE 10/2/2008

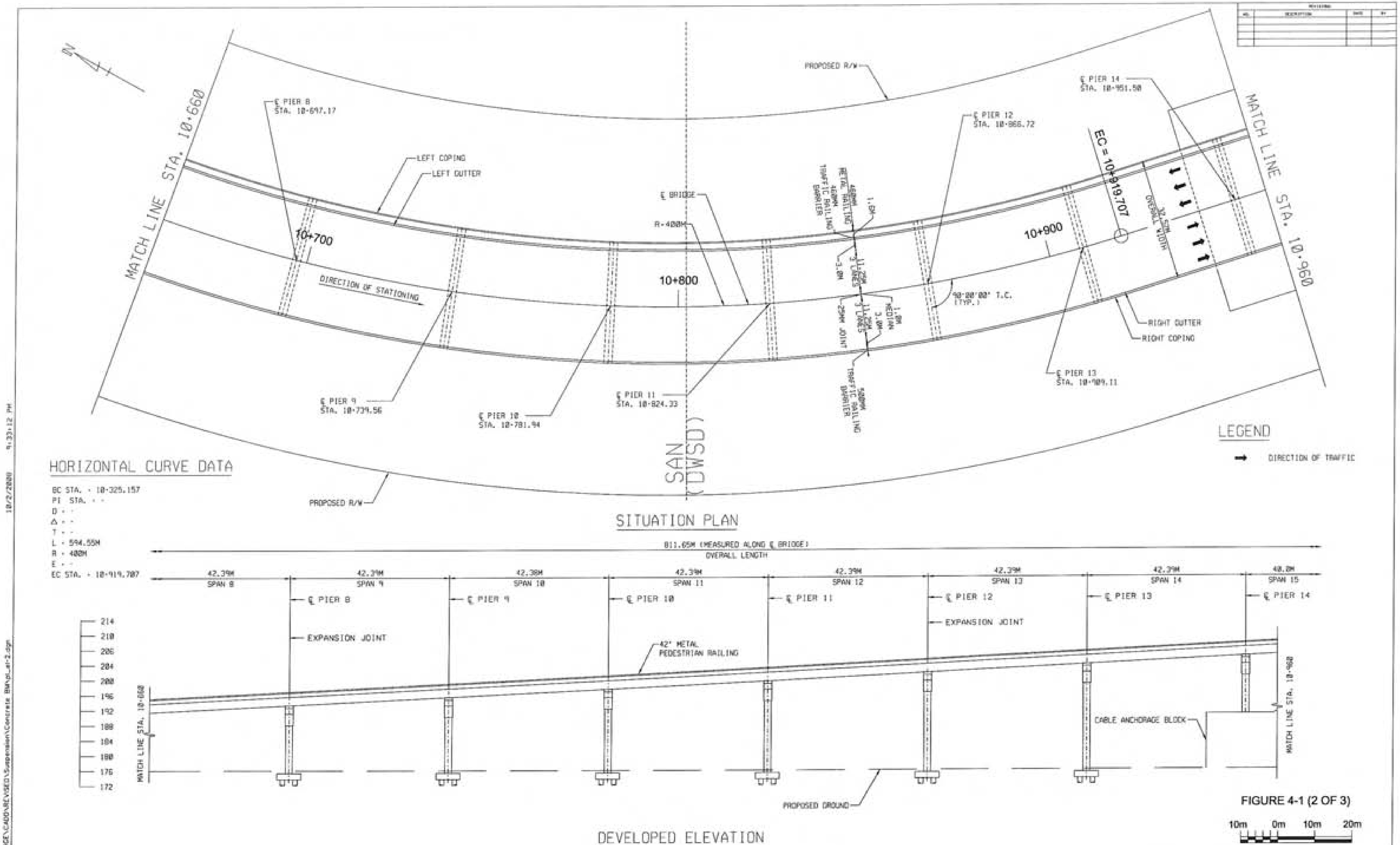


FIGURE 4-1 (2 OF 3)

Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE A1
PRESTRESSED CONC. BEAM (SUSPENSION BR)

SCALE 1:100
DATE 10/2/2008

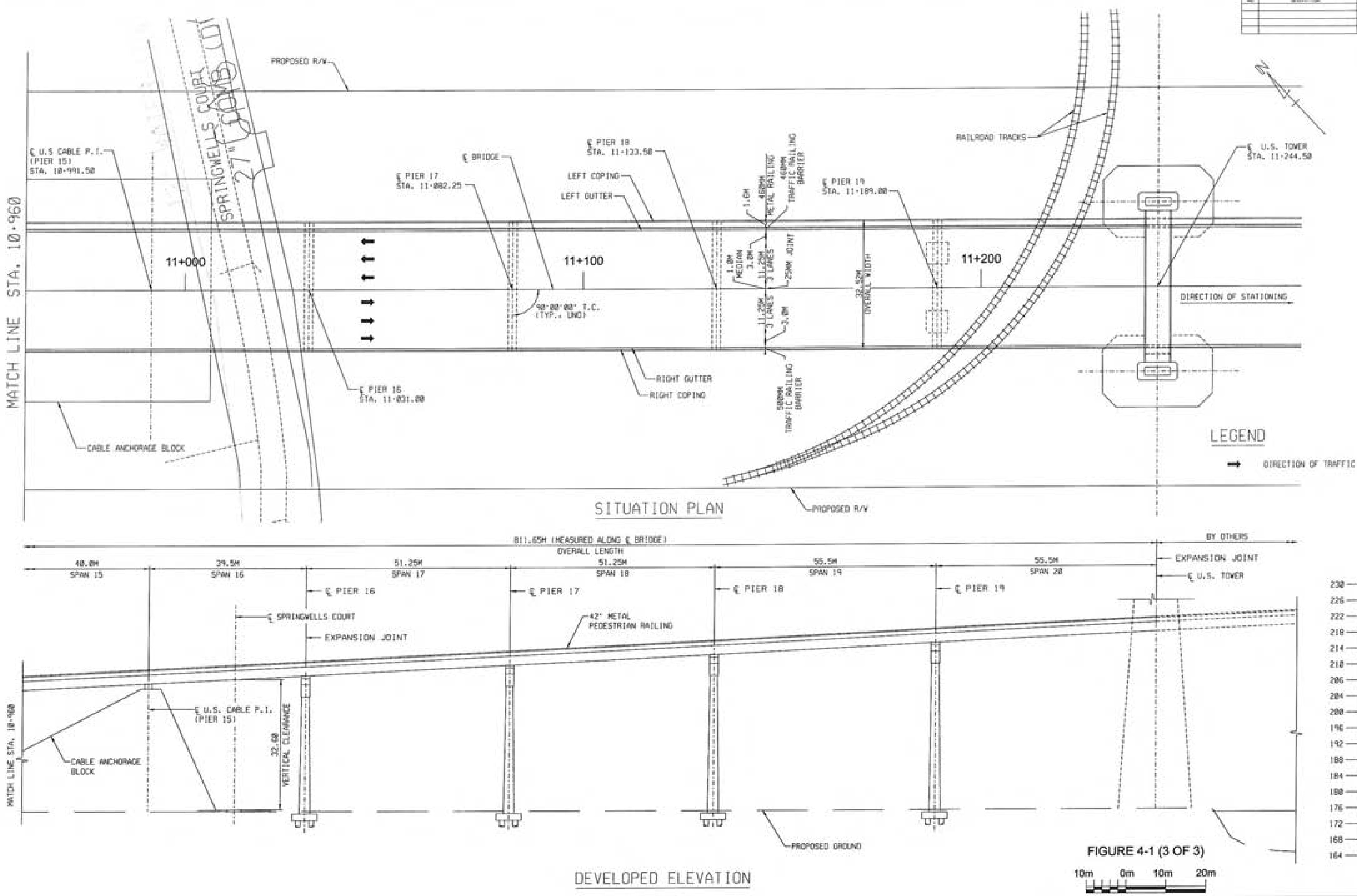


FIGURE 4-1 (3 OF 3)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE A1
PRESTRESSED CONC. BEAM (SUSPENSION BR)

SCALE 1:100
DATE 10/2/2008

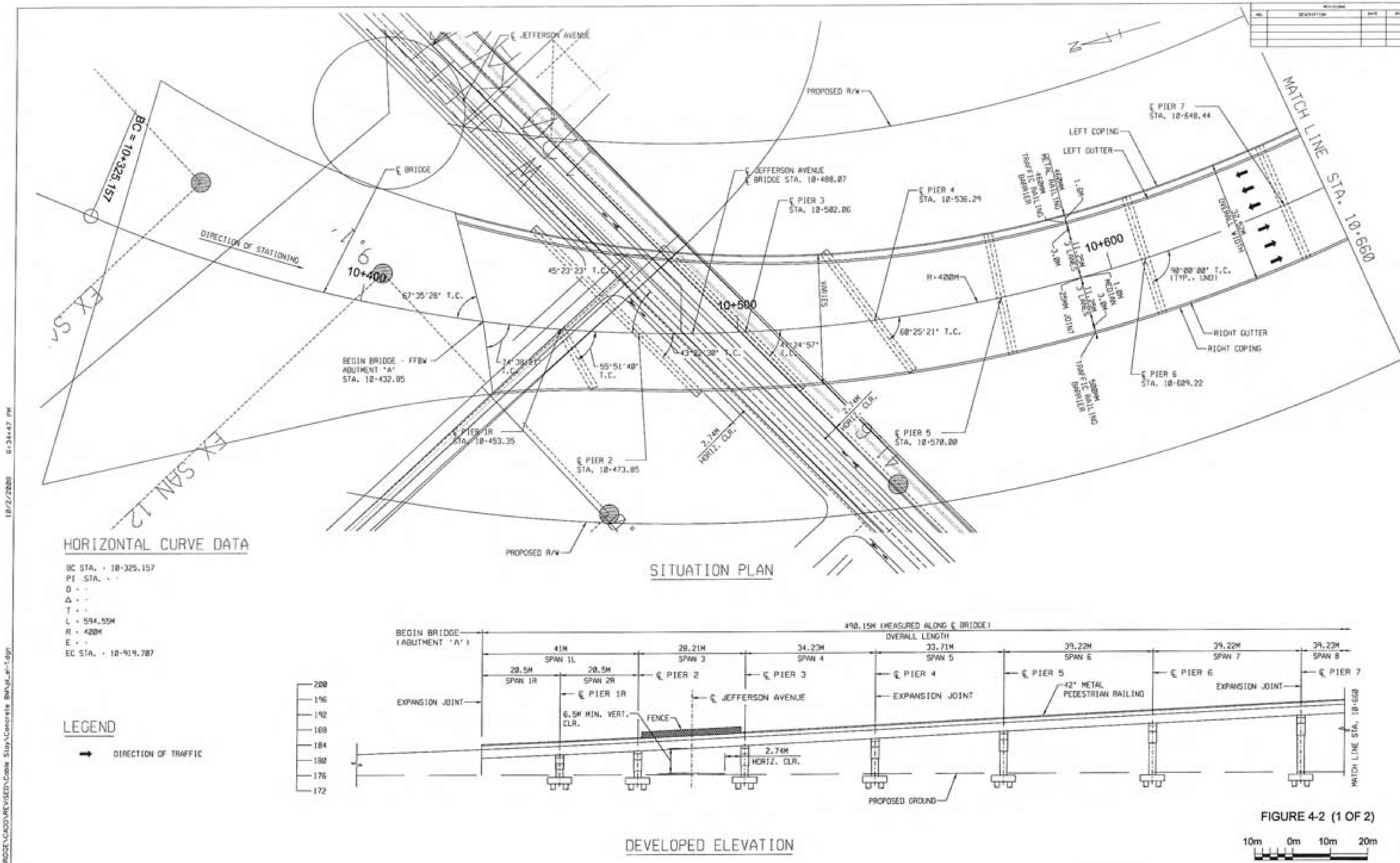


FIGURE 4-2 (1 OF 2)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

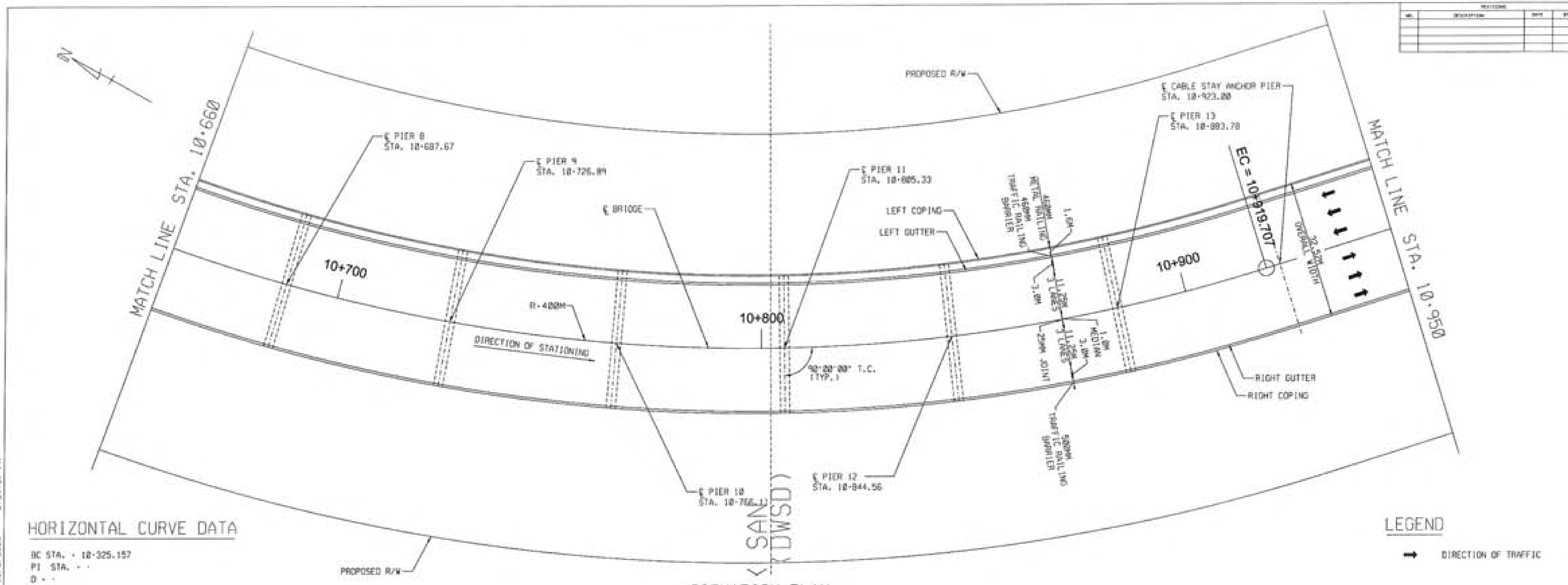
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE A2
PRESTRESSED CONC. BEAMS (CABLE STAY)

SCALE 1:100

DATE 10/2/2008



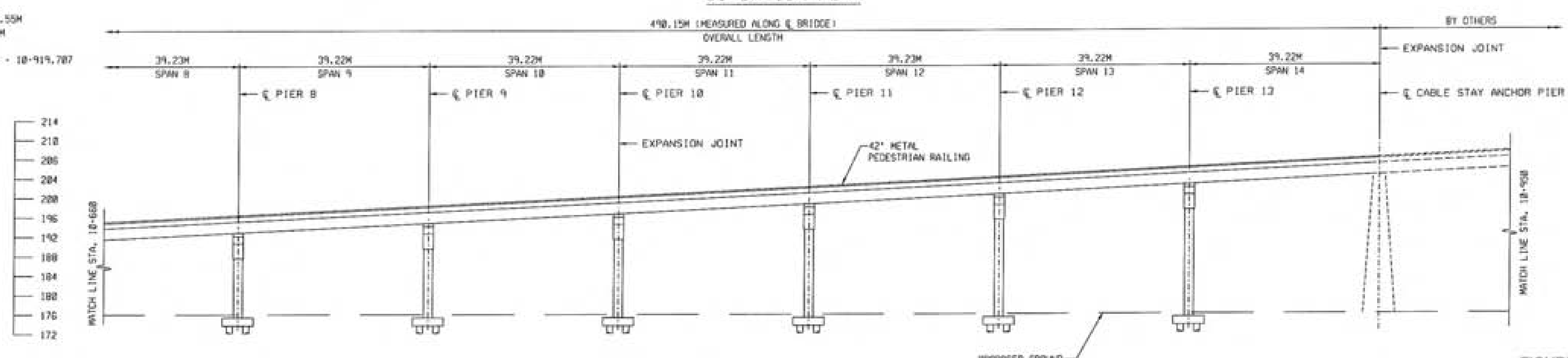
SITUATION PLAN

HORIZONTAL CURVE DATA

BC STA. = 10+325.157
PI STA. = -
D = -
Δ = -
T = -
L = 594.55M
R = 400M
E = -
EC STA. = 10+919.707

LEGEND

→ DIRECTION OF TRAFFIC



DEVELOPED ELEVATION

FIGURE 4-2 (2 OF 2)

10m 0m 10m 20m

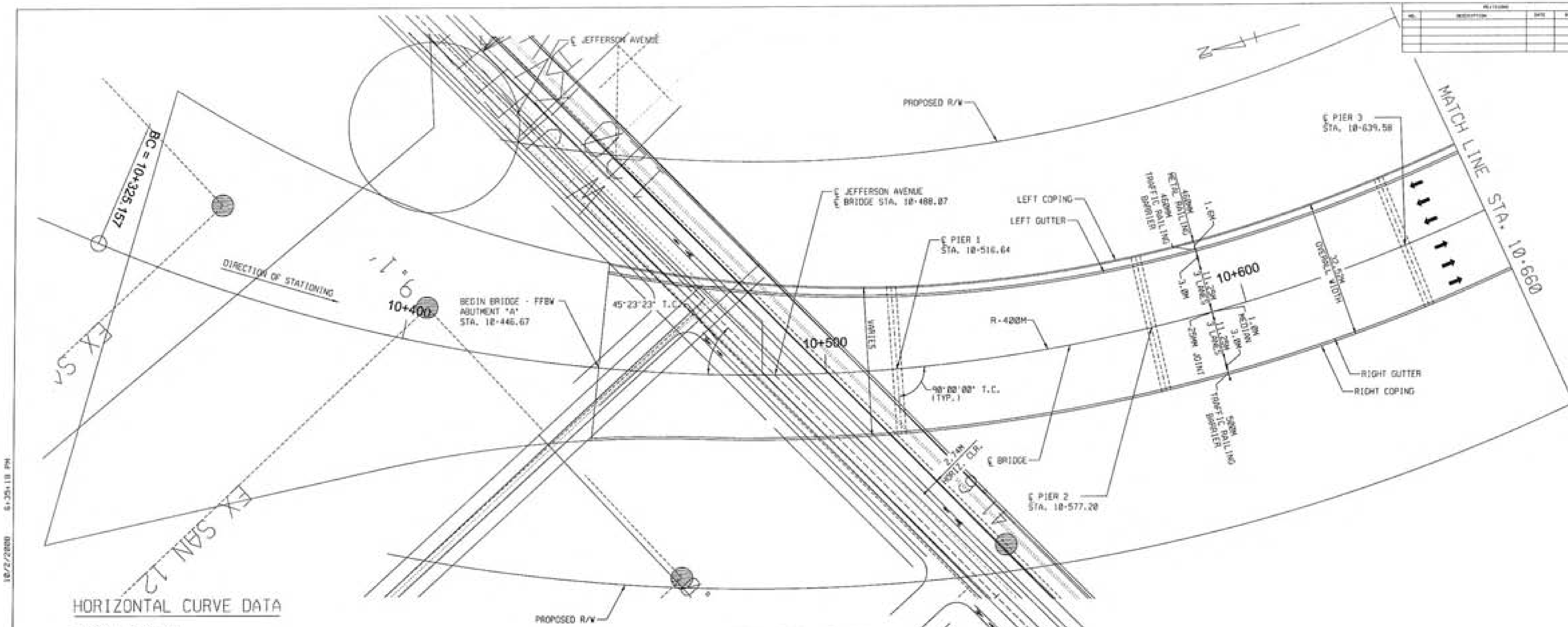
U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE A2
PRESTRESSED CONC. BEAMS (CABLE STAY)

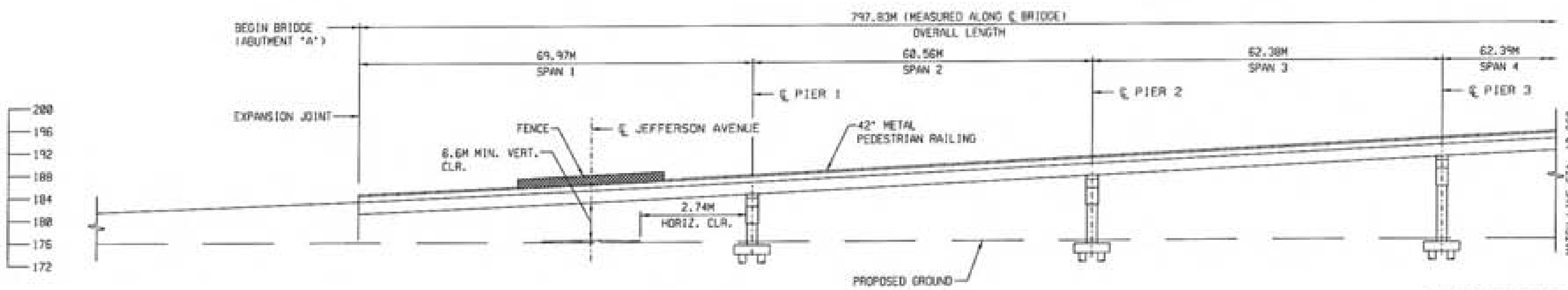
SCALE 1:100
DATE 10/2/2008



SITUATION PLAN

HORIZONTAL CURVE DATA

BC STA. = 10+325.157
PI STA. = -
D = -
Δ = -
T = -
L = 594.55M
R = 400M
E = -
EC STA. = 10+919.707



DEVELOPED ELEVATION

LEGEND

➡ DIRECTION OF TRAFFIC

FIGURE 4-3 (1 OF 3)

10m 0m 10m 20m

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario	DETROIT RIVER INTERNATIONAL CROSSING	PARSONS	SITUATION PLAN - ALTERNATE B1 STEEL PLATE GIRDER (SUSPENSION BR)

SCALE 1:100
DATE 10/2/2008

10/2/2008 6:35:18 PM

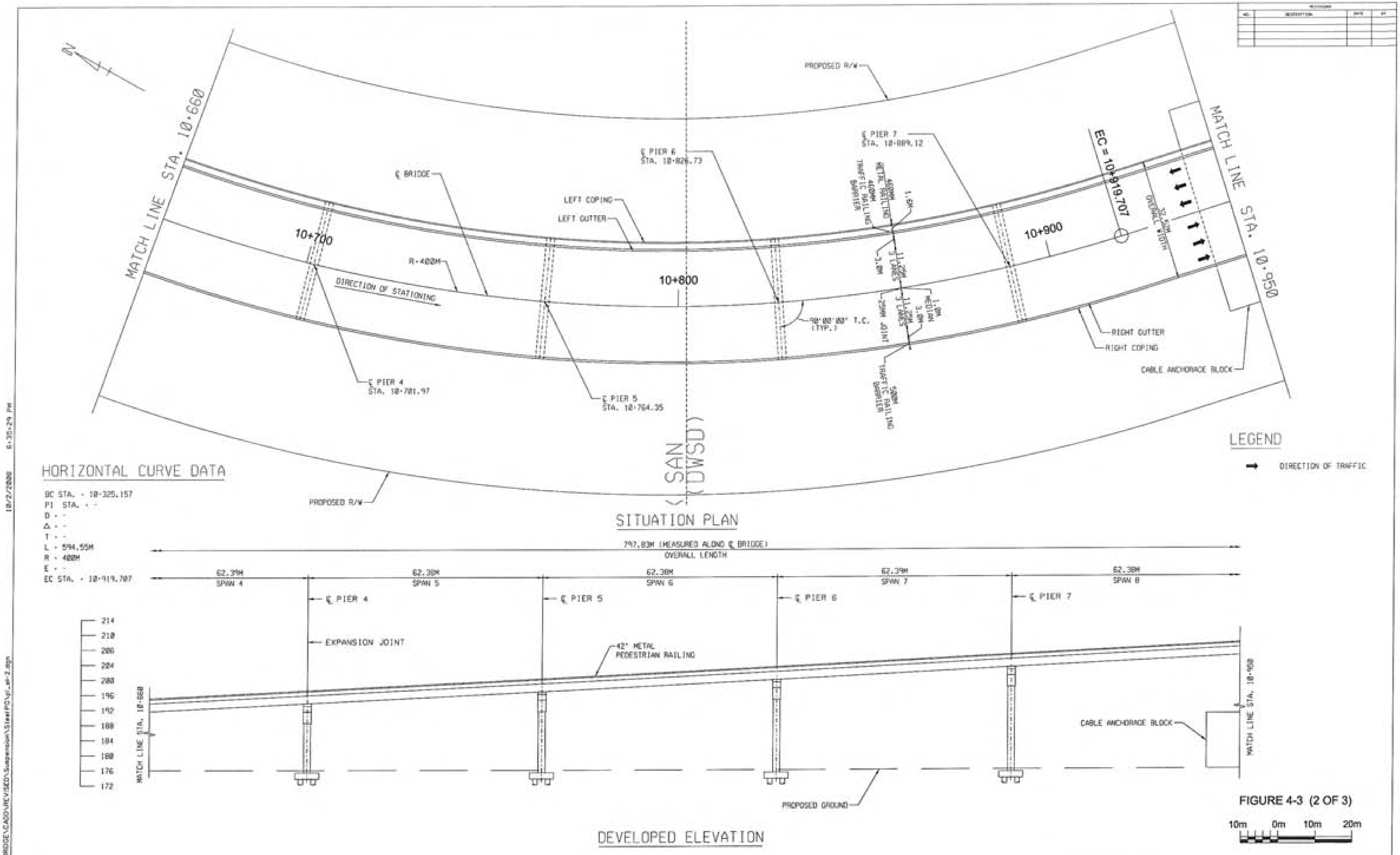


FIGURE 4-3 (2 OF 3)

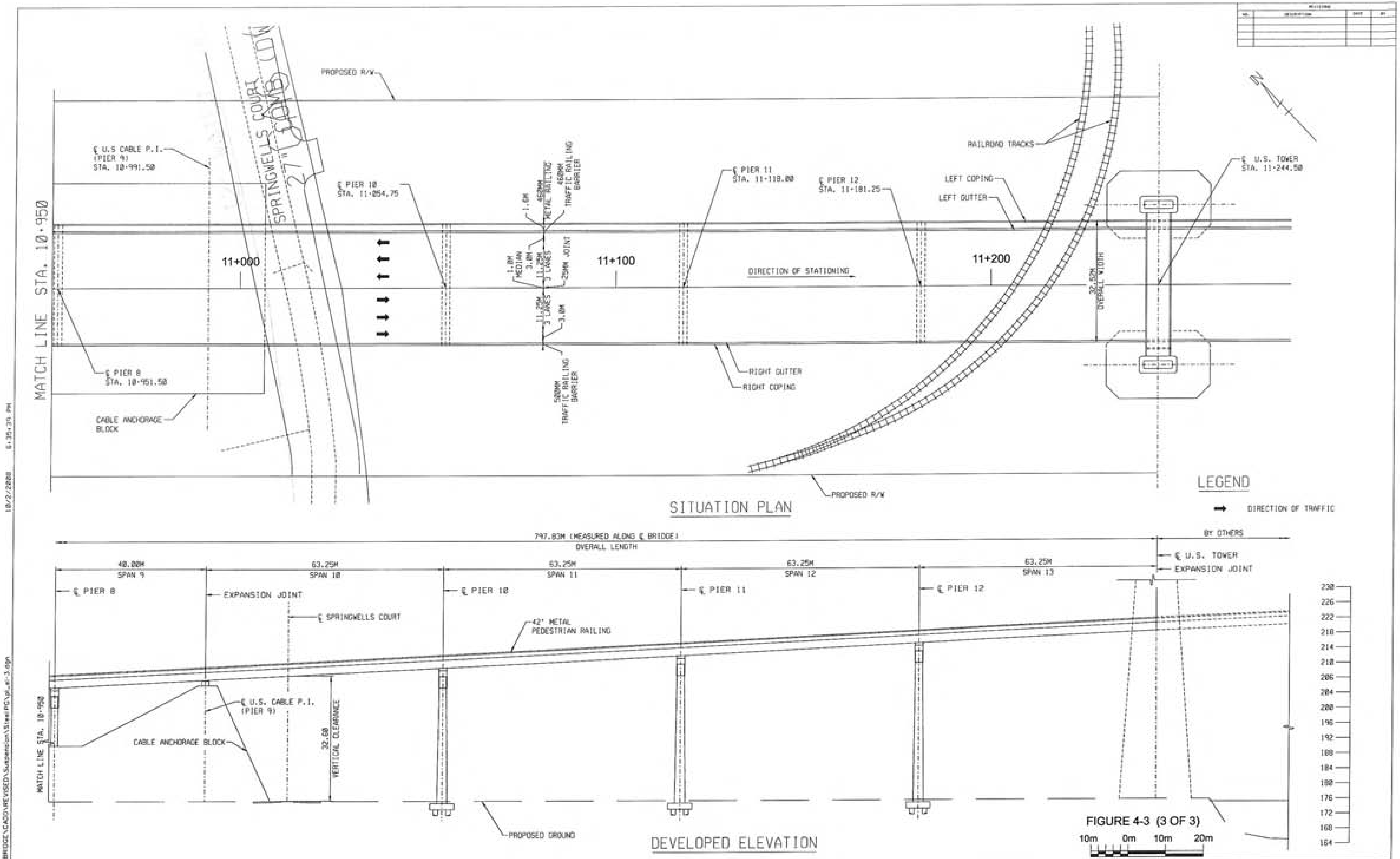
U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE B-1
STEEL PLATE GIRDER (SUSPENSION BR)

SCALE 1:100
DATE 10/2/2008



U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE B1
STEEL PLATE GIRDER (SUSPENSION BR)

SCALE 1:100
DATE 10/2/2008

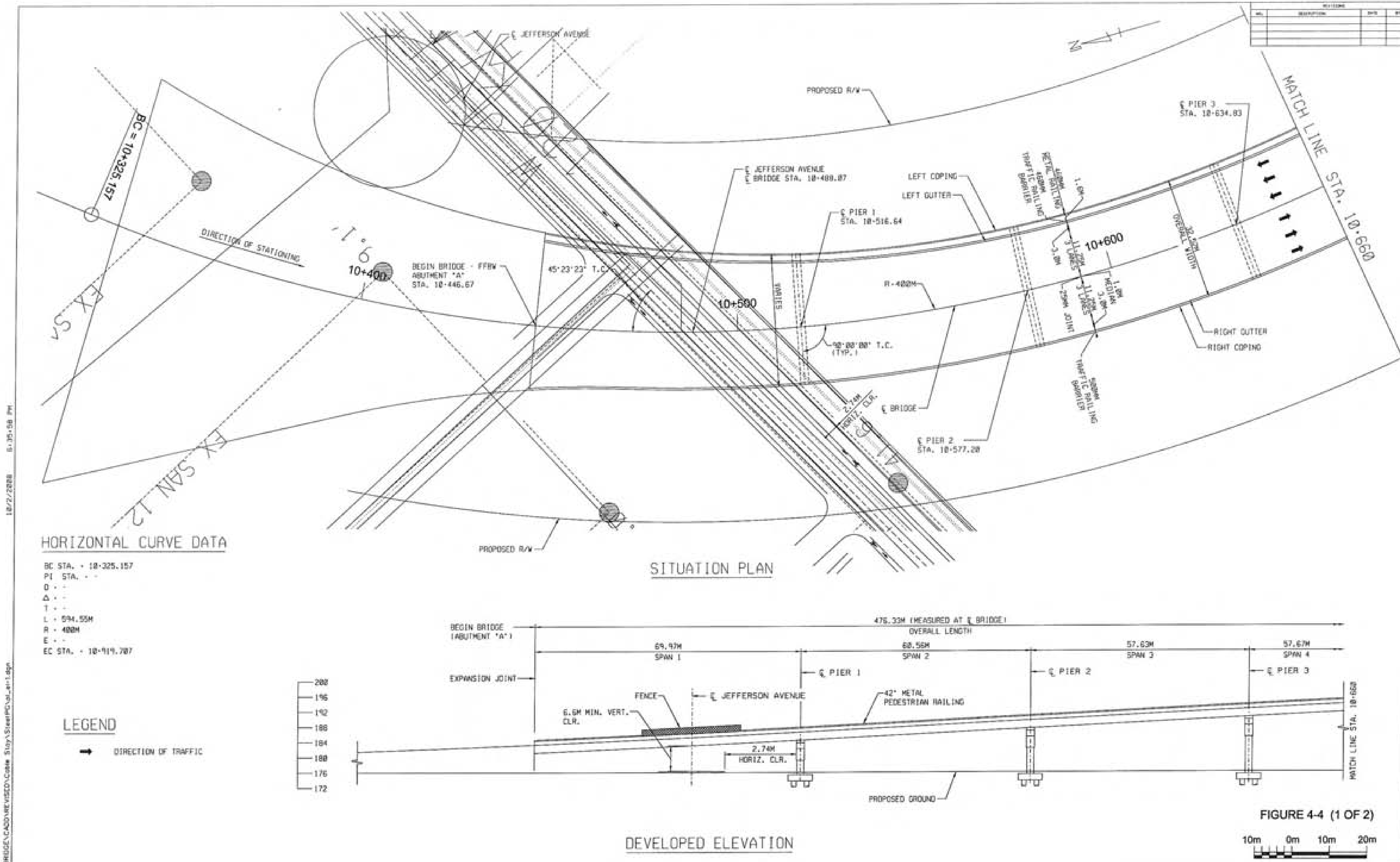


FIGURE 4-4 (1 OF 2)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE B2
STEEL PLATE GIRDER (CABLE STAY)

SCALE 1:100

DATE 10/2/2008

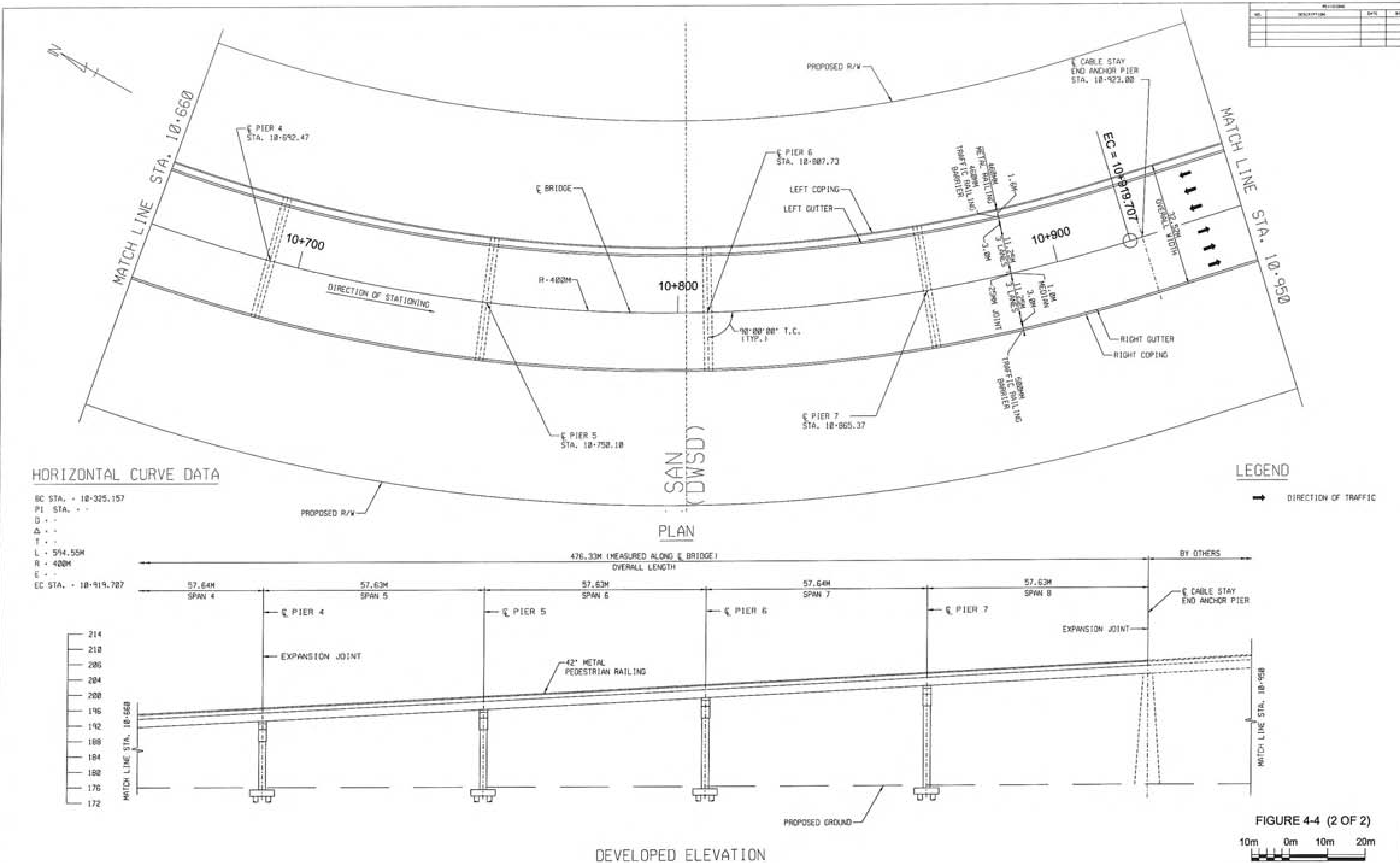


FIGURE 4-4 (2 OF 2)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

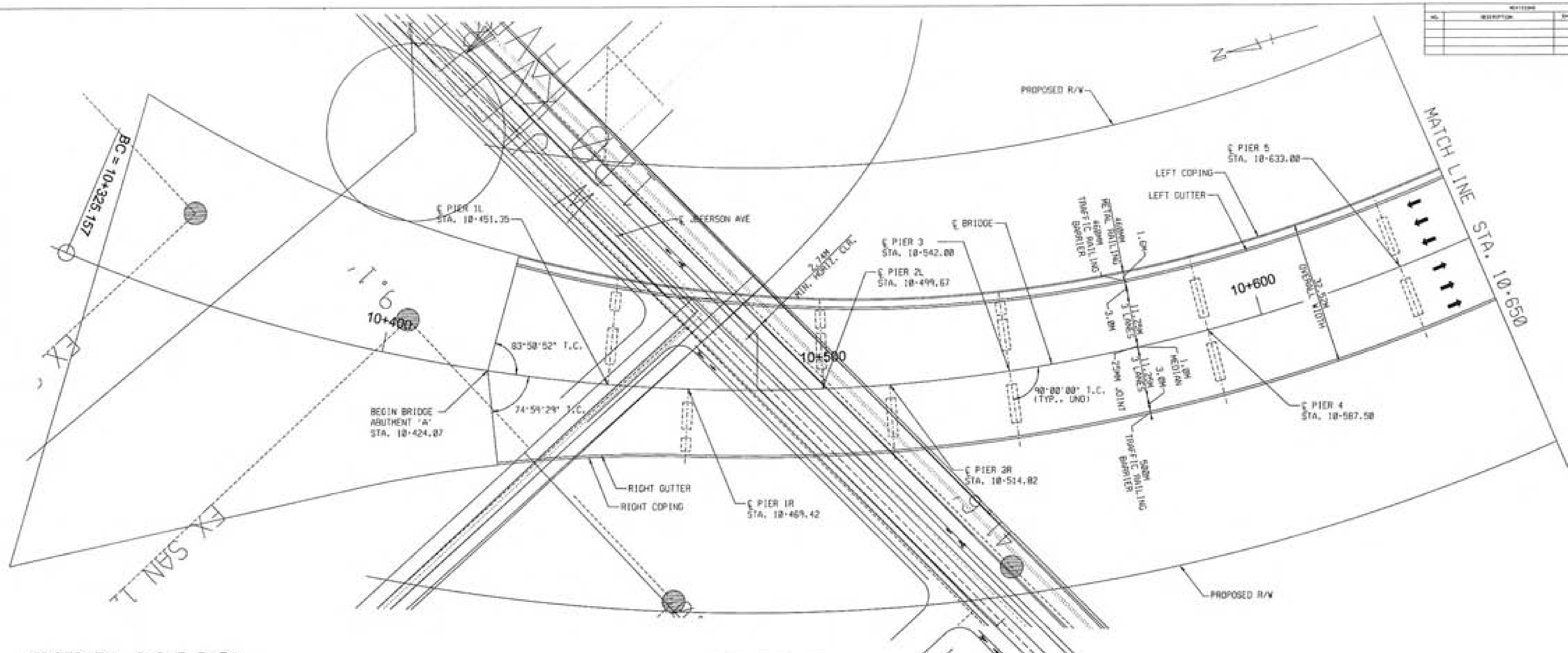
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE B2 STEEL PLATE GIRDER (CABLE STAY)

SCALE 1:100

DATE 10/2/2008



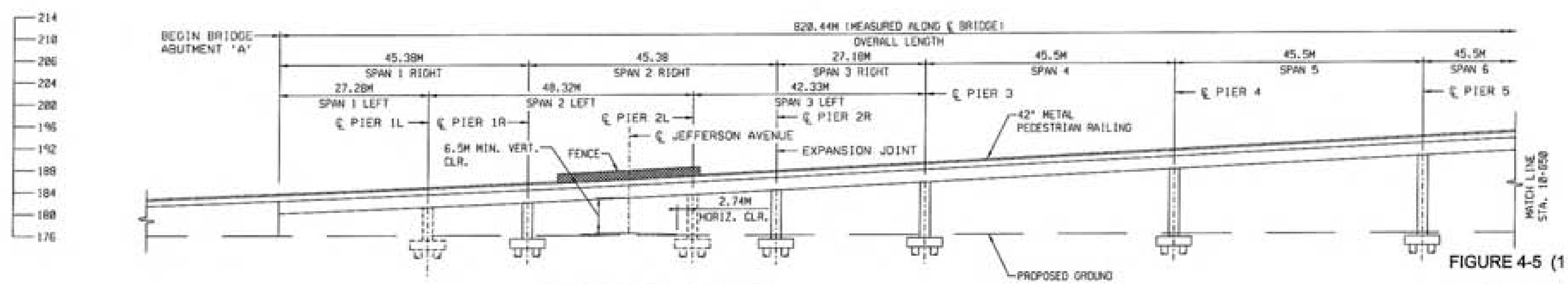
SITUATION PLAN

HORIZONTAL CURVE DATA

BC STA. = 10+325.157
PI STA. = -
D = -
Δ = -
T = -
L = 594.55M
R = 400M
E = -
EC STA. = 10+919.707

LEGEND

➡ DIRECTION OF TRAFFIC



DEVELOPED ELEVATION

FIGURE 4-5 (1 OF 3)

10m 0m 10m 20m

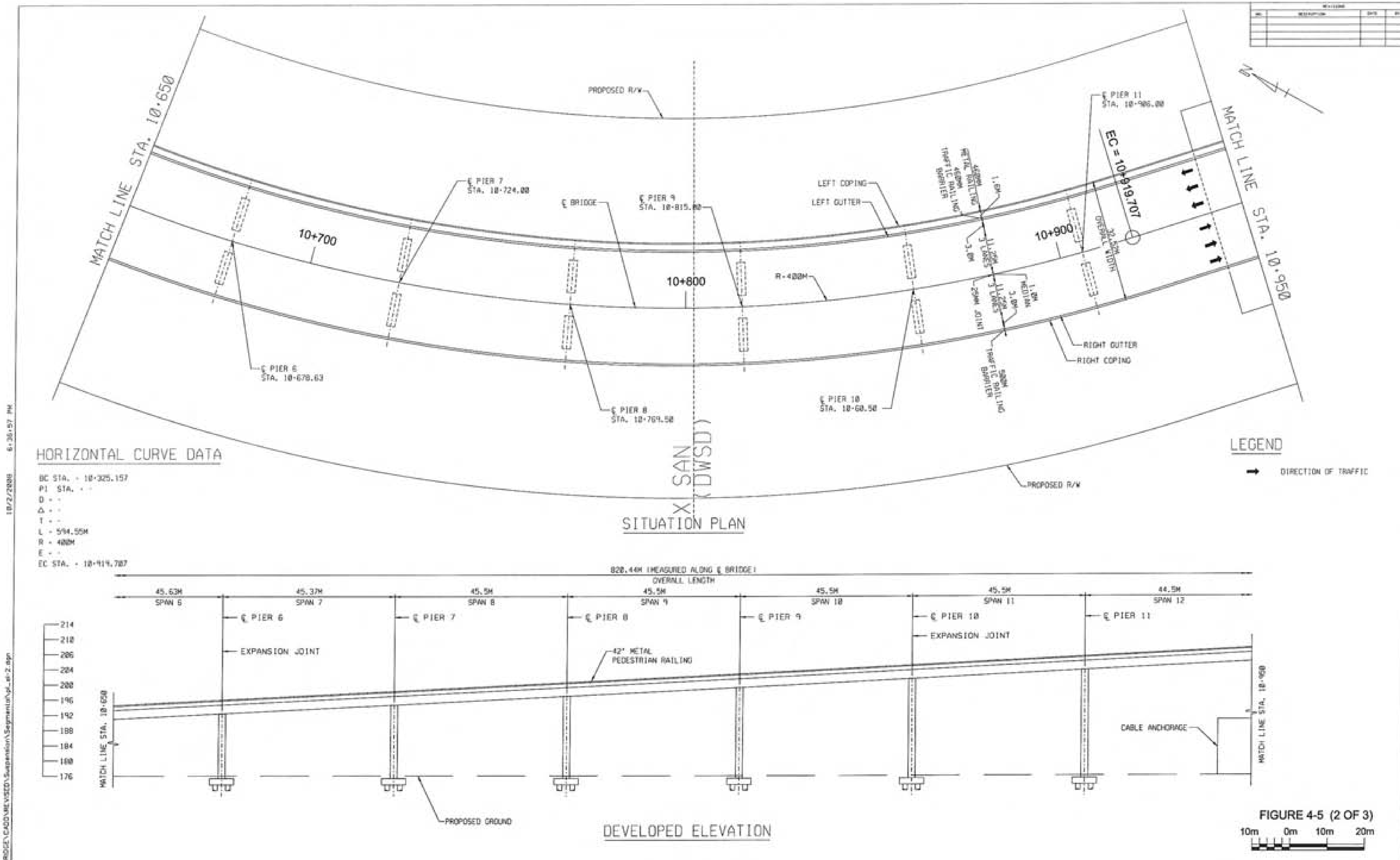
U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE C1
SEGMENTAL BEAMS (SUSPENSION)

SCALE 1:100
DATE 10/2/2008



U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE C1
SEGMENTAL BEAMS (SUSPENSION)

SCALE 1:100

DATE 10/2/2008

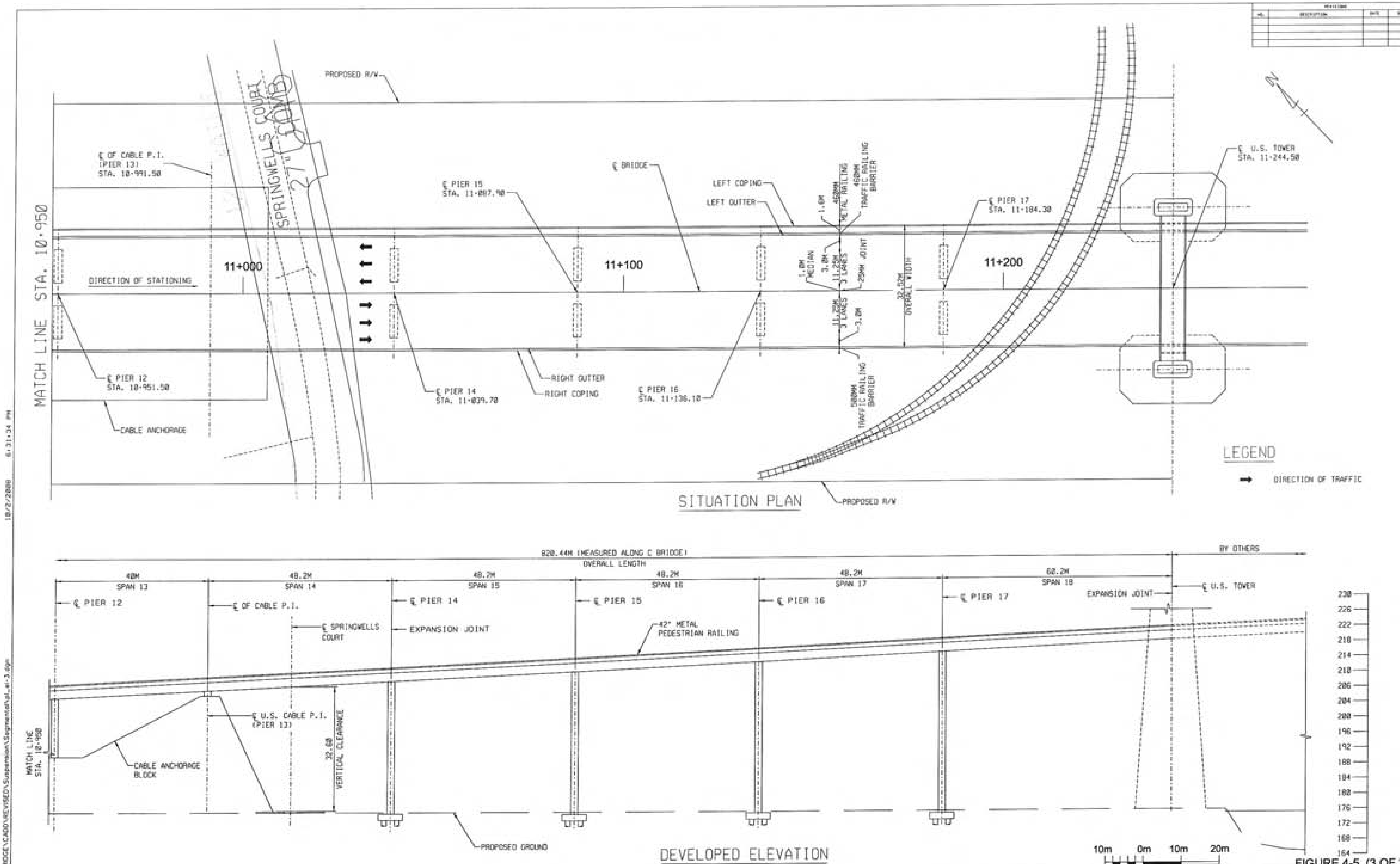


FIGURE 4-5 (3 OF 3)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

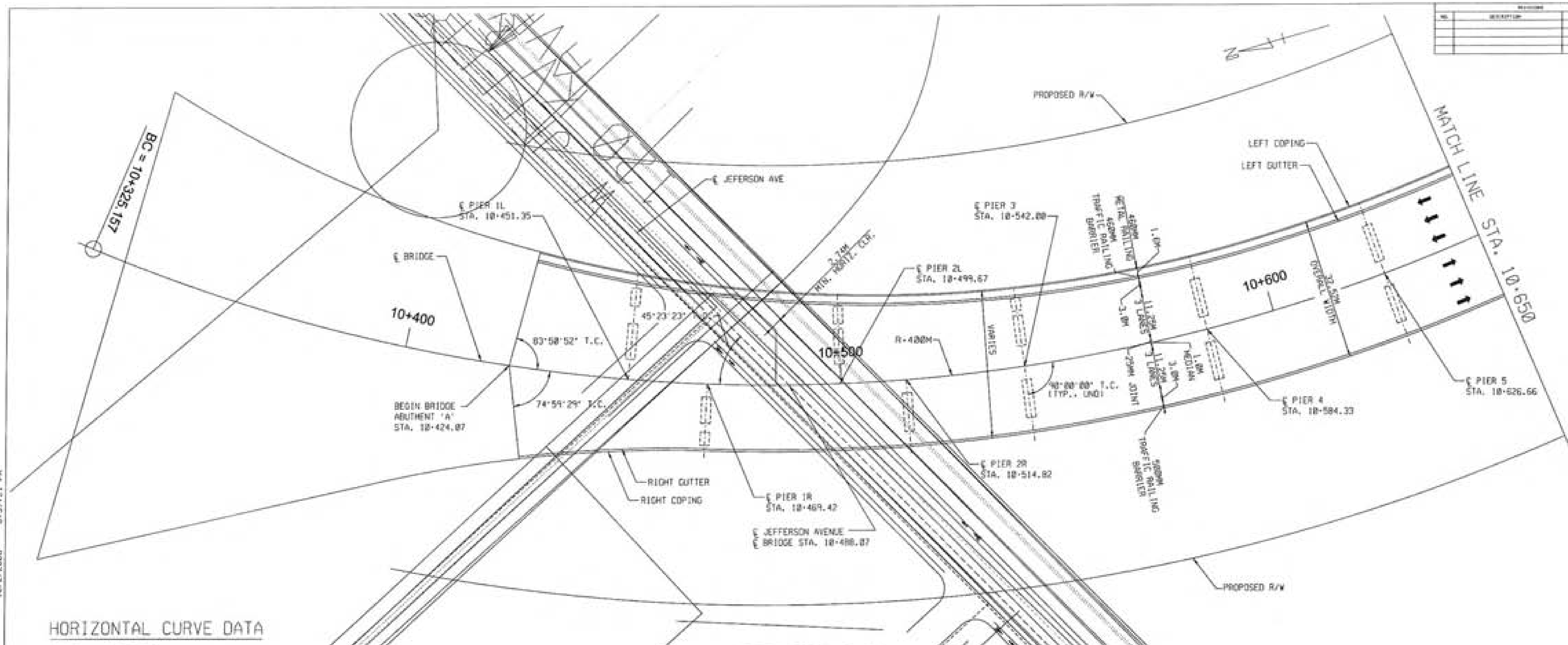
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE C1
SEGMENTAL BEAMS (SUSPENSION)

SCALE 1:100

DATE 10/2/2008



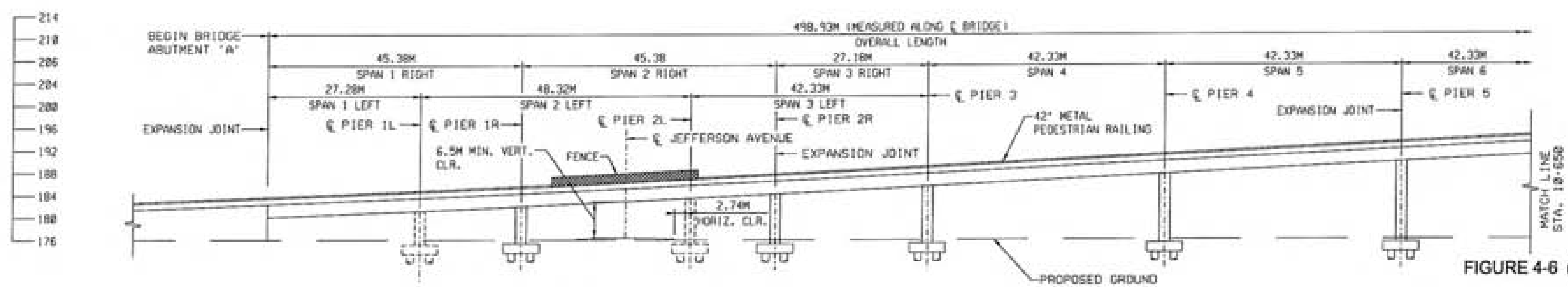
SITUATION PLAN

HORIZONTAL CURVE DATA

BC STA. = 10+325.157
PI STA. = -
D = -
Δ = -
T = -
L = 594.55M
R = 400M
E = -
EC STA. = 10+919.707

LEGEND

➡ DIRECTION OF TRAFFIC



DEVELOPED ELEVATION

FIGURE 4-6 (1 OF 2)

10m 0m 10m 20m

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE C2
SEGMENTAL BEAMS (CABLE STAY)

SCALE 1:100

DATE 10/2/2008

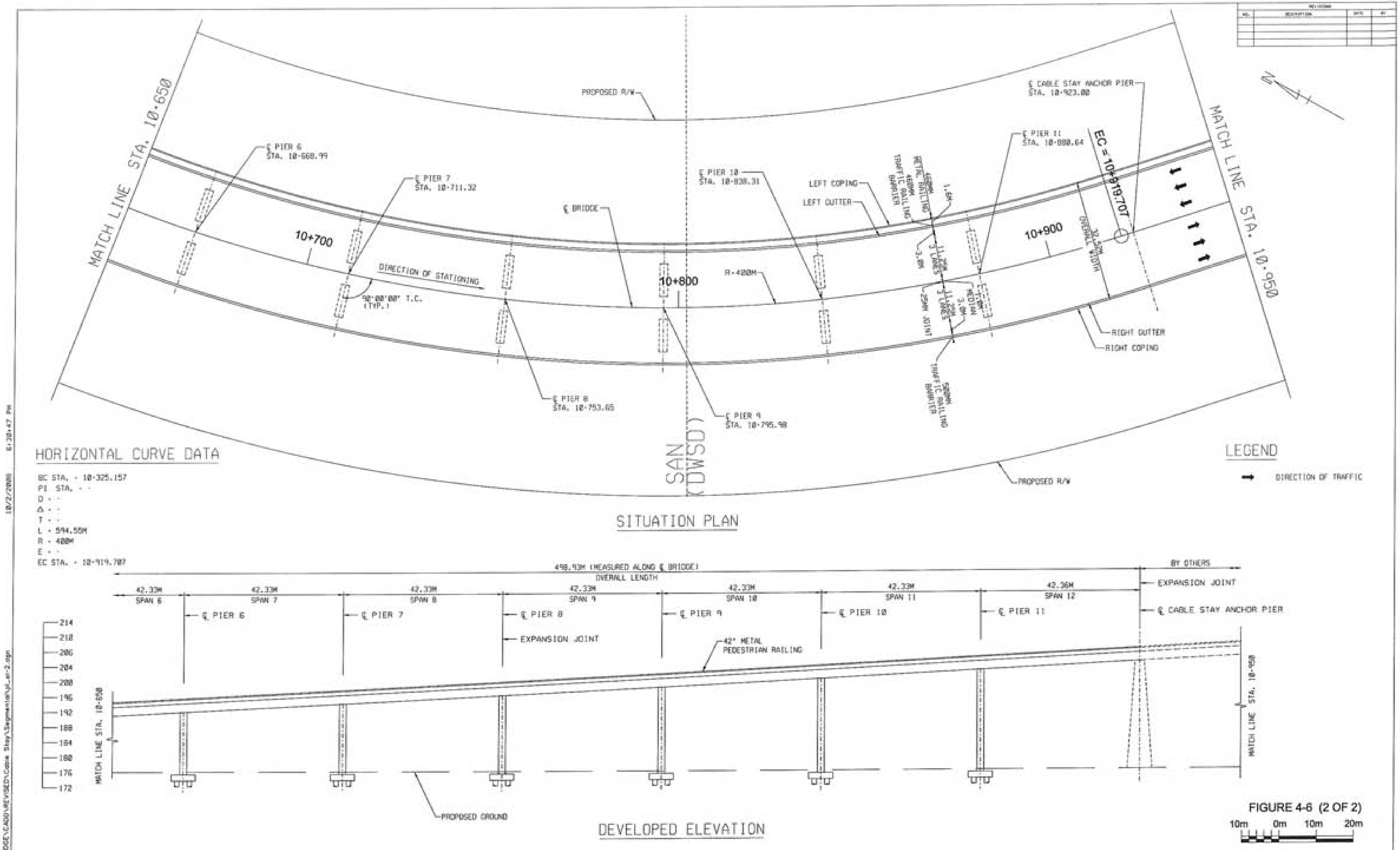


FIGURE 4-6 (2 OF 2)

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT · Ontario

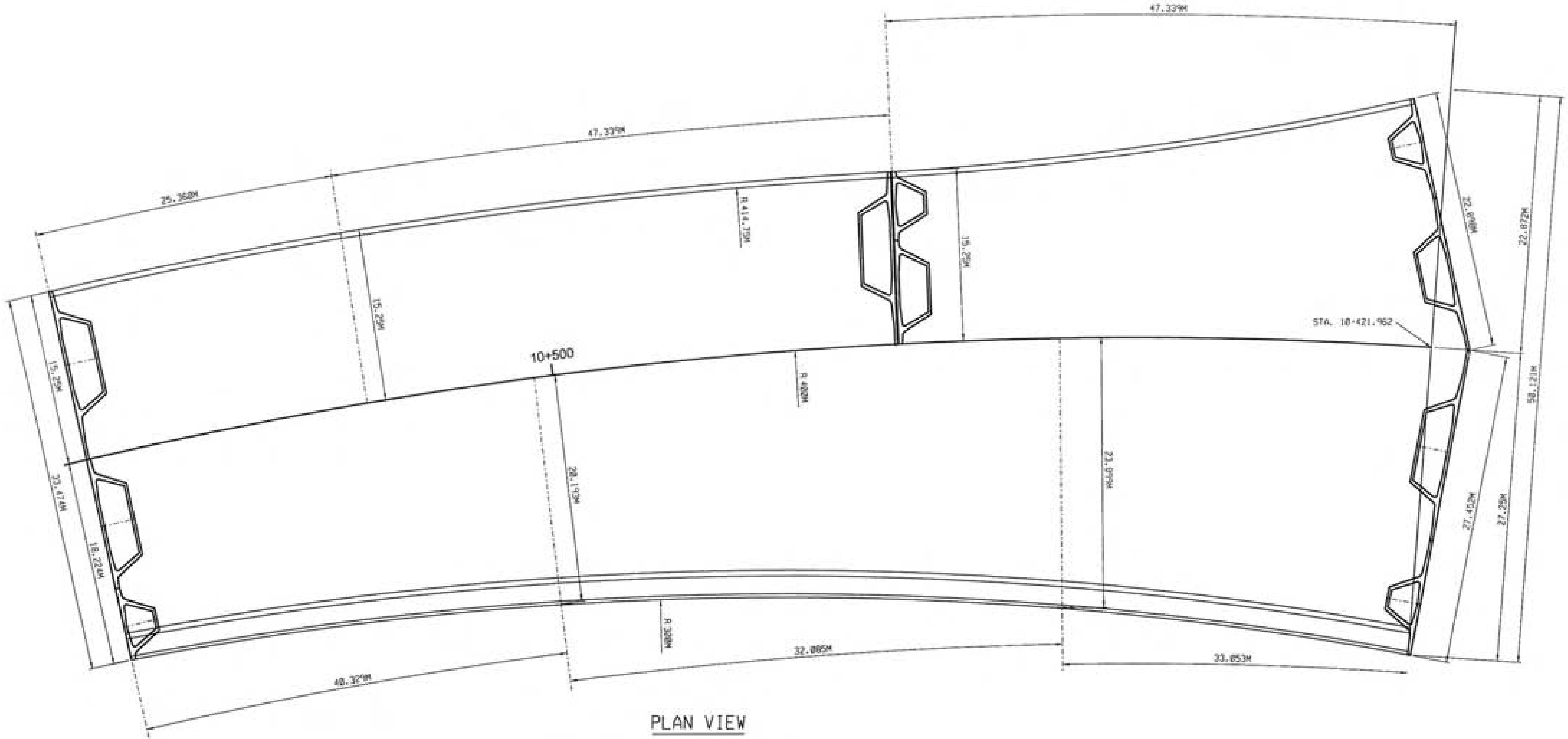
DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

SITUATION PLAN - ALTERNATE C2
SEGMENTAL BEAMS (CABLE STAY)

SCALE 1:100

DATE 10/2/2008



PLAN VIEW

VARIABLE WIDTH DECK - SEGMENTAL

FIGURE 4-7

U.S. Department of Transportation Federal Highway Administration · Canada · MDOT Michigan Department of Transportation · Ontario

DETROIT RIVER INTERNATIONAL CROSSING

PARSONS

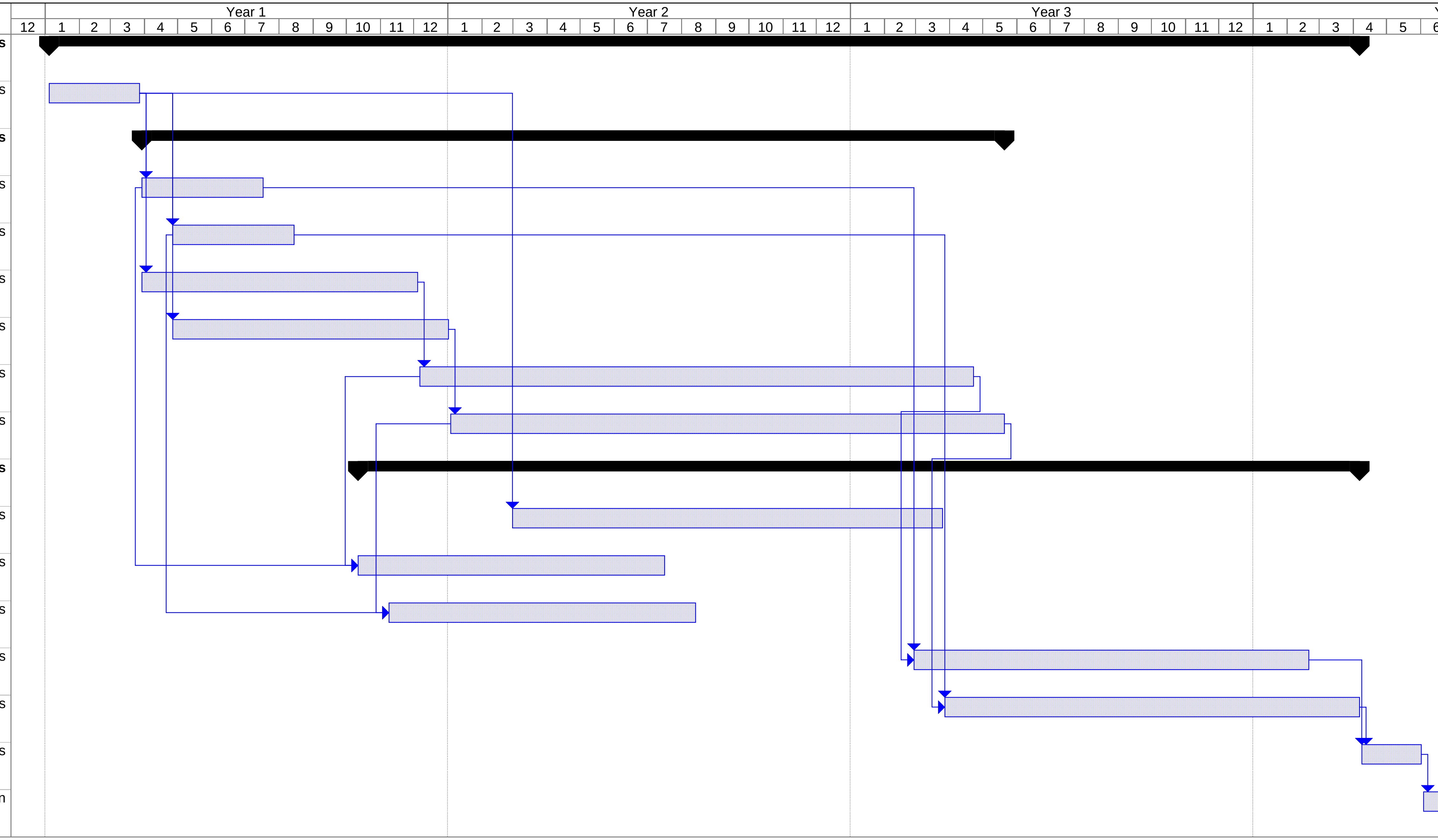
SEGMENTAL BOX GIRDER
SPLAYED DECK ARRANGEMENT

SCALE	1:100
DATE	10/2/2008

APPENDIX B – Construction Schedules

OPTION 4, CONCEPTUAL ENGINEERING SCHEDULE

ID	Task Name	Duration
1	**TS Option 4 Construction**	**850 days**
2	Mobilization	3 mons
3	**Substructure**	**560 days**
4	N. Anchor Pier	4 mons
5	S. Anchor Pier	4 mons
6	N. Tower Fdn	9 mons
7	S. Tower Fdn	9 mons
8	North Tower	18 mons
9	South Tower	18 mons
10	**Superstructure**	**650 days**
11	Deck Fabrication	14 mons
12	North Cast-in-place backspan	10 mons
13	South Cast-in-place backspan	10 mons
14	Stay and Deck Erection - North Tower	256 days
15	Stay and Deck Erection - South Tower	270 days
16	Final Finishing Work	2 mons
17	Demobilization	1 mon

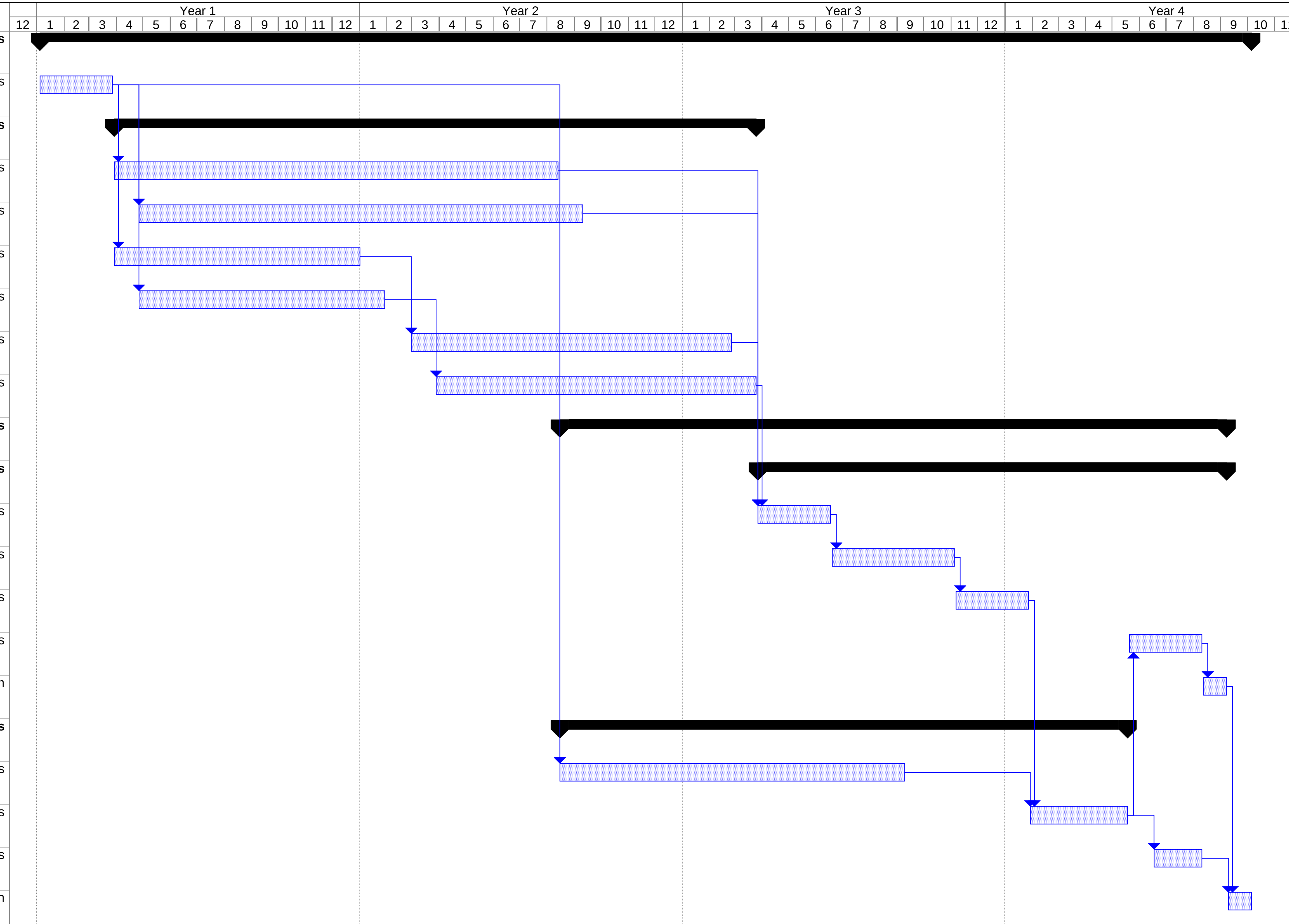


Project: Suspension Bridge TS#1 v1 r:
Date: Mon 11/5/07

Task
Split
Progress
Milestone
Summary
Project Summary
External Tasks
External Milestone
Deadline

OPTION 7, CONCEPTUAL ENGINEERING SCHEDULE

ID	Task Name	Duration
1	**CE Option 7 Susp. Bridge Construction**	**980 days**
2	Mobilization	3 mons
3	**Substructure**	**520 days**
4	N. Anchorage	18 mons
5	S. Anchorage	18 mons
6	N. Tower Fdn	10 mons
7	S. Tower Fdn	10 mons
8	North Tower	13 mons
9	South Tower	13 mons
10	**Superstructure**	**540 days**
11	**Main Cable - In Place**	**380 days**
12	Equipment Erection, Catwalk	3 mons
13	Cable Spinning & compacting	5 mons
14	Bands & Suspenders	3 mons
15	Cable Wrapping	3 mons
16	Remove Catwalk	1 mon
17	**Deck**	**460 days**
18	Fabrication	14 mons
19	Deck Erection	4 mons
20	Finishing Work	2 mons
21	Demobilization	1 mon



Project: DRIC Susp. Bridge Const.
Date: Mon 11/5/07

Task | Progress | Summary | External Tasks | Deadline
Split | Milestone | Project Summary | External Milestone

APPENDIX C – Detailed Cost Estimates

PARSONS

Project DRIC | Job no. 646294
by ____ date 22-Nov-08 | Subject Engineering Report Cost Estimate
chk ____ date ____
###

OPTION: PA Option 4 — CROSSING: X-10(B)

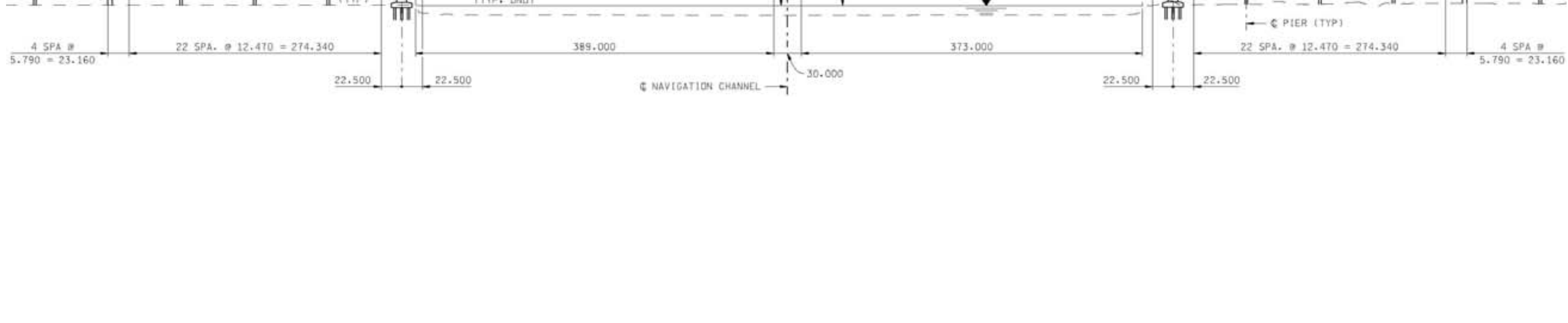
Geometry

			Canada	431.06	
			US	499.15	
North Abutment:	STA	10+423.85	Main Span Length	840	meters
North End of Suspended Spans:	STA	10+923.00	Suspended Span Length	1480	meters
North Pylon:	STA	11+243.00	US Approach Span	499	meters
South Pylon:	STA	12+083.00	CAN Approach Span	431	meters
South End of Suspended Spans:	STA	12+403.00	Orthotropic Deck Length	820	meters
South Abutment:	STA	12+834.06	Concrete Deck Length	660	meters
Length of Suspended Spans		1480 meters	Cable-to-Cable Width	34.73	meters
Length of Approaches		930 meters	Curb-to-Curb Width	31.52	meters

CABLE STAY BRIDGE — **Inverted Y Pylon**

PREPARED BY: Y.ARONSON / N.VOLDMAN

Item	Revised Unit QTY	QTY Mult.	QTY Total	Unit Cost	Total	Total
Superstructure						
Deck						
Orthotropic Box Girder	438.2 kg/m^2	28,479 m^2	*12,479,715* kg	$ 11.43 USD/kg	$ 142,643,142	
Concrete Box Girder Including shoring	39.3 m^3/m	660 m	*25,938* m^3	$ 2,361.55 USD/m3	$ 61,253,801	
Box Girder Reinforcing	130.5 kg/m^3	25,938 m^3	*3,384,909* kg	$ 2.91 USD/kg	$ 9,850,085	
Box Girder Postensioning	29.5 kg/m^3	25,938 m^3	*765,171* kg	$ 8.54 USD/kg	$ 6,534,560	
Security/Hardening Measures (5% of above)					$ 11,014,079	
Deck Subtotal					*$ 231,295,668*	
Stay System						
Stay Cables	1,661,054 kg/plane	2 plane	*3,322,107* kg	$ 12.52 USD/kg	$ 41,592,780	
Deck Anchorages	1,178 kg/stay	216 stays	*254,545* kg	$ 6.75 USD/kg	$ 1,718,182	
Damping Devices	1 ea/stay	216 stays	*216* ea	in above		
Security/Hardening Measures (5% of above)					$ 2,165,548	
Stay System Subtotal					*$ 45,476,509*	
Misc. Appurtenances						
Overlay, Barriers and membrane	51,400 m^2/br	1 br	51,400 m^2	$ 300.00 USD/m2	$ 15,420,120	
Lighting , Drainage, etc.	51,400 m^2/br	1 br	51,400 m^2	$ 100.00 USD/m2	$ 5,140,040	
Fiber Optic Cable		1 br	1,480 m	$ 147.60 USD/m	$ 218,448	
Fiber Optic Cable Node		1 br	2 ea	$ 110,000.00 USD/ea	$ 220,000	
Miscellaneous Items	51,400 m^2/br	1 br	51,400 m^2	$ 120.00 USD/m2	$ 6,168,048	
Misc. Appurtenances Subtotal					*$ 27,166,656*	
Superstructure Subtotal						***$ 303,939,000***
Substructure (Cost Includes dewatering and excavation)						
Pylons						
Concrete	34.8 m^3/m	*499* m for 2	*17,365* m^3	$ 3,219.81 USD/m3	$ 55,912,722	
Reinforcing Steel	237.3 kg/m^3	17,365 m^3	*4,120,762* kg	$ 2.91 USD/kg	$ 11,991,417	
Strut Post Tensioning	30.0 kg/m^3	1,260 m^3	*37,800* kg	$ 8.54 USD/kg	$ 322,812	
Structural Steel	1,389 kg/stay	216 stays	*300,000* kg	$ 11.43 USD/kg	$ 3,429,000	
Stairs and Elevators for Access					$ 8,000,000	
Security/Hardening Measures (5% of above)					$ 3,982,798	
Pylons Subtotal					*$ 83,638,749*	
Pylon Foundations						
Footing Concrete	6,132 m^3/ea	*2* ea	*12,264* m^3	$ 588.69 USD/m3	$ 7,219,652	
Footing Reinforcing	90.0 kg/m^3	12,264 m^3	*1,103,760* kg	$ 2.91 USD/kg	$ 3,211,942	
Drilled shaft including casing	32 shafts	*150* m3/shaft	*4,800* m^3	$ 1,774.54 USD/m3	$ 8,517,792	
Rock Socket	32 shafts	*8* m3/shaft	*256* m^3	$ 3,408.13 USD/m3	$ 872,481	
Environmental Remediation			*1* ea	$ 2,000,000.00 USD/ea	$ 2,000,000	
Pylon Foundations Subtotal					*$ 21,821,867*	
Anchor Piers						
Anchor Pier Concrete	956.5 m^3/ea	*8* ea	*7,652* m^3	$ 1,705.97 USD/m3	$ 13,054,081	
Anchor Pier Reinforcing	157.0 kg/m^3	7,652 m^3	*1,201,364* kg	$ 2.91 USD/kg	$ 3,495,969	
Footing Concrete	183.5 m^3/ea	8 ea	*1,468* m^3	$ 917.47 USD/m3	$ 1,346,849	
Footing Reinforcing	90.0 kg/m^3	1,468 m^3	*132,120* kg	$ 2.91 USD/kg	$ 384,469	
Drilled Shaft (1.219mx30m ø / pier),incl.casing 16 mm	80 shafts	*35* m3/shaft	*2,800* m^3	$ 1,774.54 USD/m3	$ 4,968,712	
Rock Socket	80 shafts	*2* m3/shaft	*160* m^3	$ 3,408.13 USD/m3	$ 545,301	
Environmental Remediation			*4* ea	$ 150,000.00 USD/ea	$ 600,000	
Security/Hardening Measures (2% of above)					$ 487,908	
Anchor Piers Subtotal					*$ 24,883,289*	
Support Facilities						
Yard & barges		1 br	1 br	$ 7,000,000 ea	$ 7,000,000	
Subtotal					*$ 7,000,000*	
Substructure (Cost Includes dewatering and excavation) Subtotal						***$ 137,344,000***
Quantities Subtotal (rounded)					***$ 441,283,000***	
General Conditions Bond and Insurance 11%					$ 48,541,130	
GC's Overhead and Profit 10%					$ 48,982,413	
Design Contingency 10%					$ 53,880,654	
Main Bridge Total (rounded)						**$ 592,687,000**
Approaches						
US Approach Bridge	15,733 m2	1 ea	15,733 m2	$ 1,804.63 USD/M2	$ 28,392,666	
Security/Hardening Measures (3% of above)					$ 851,780	
CN Approach Bridge	13,587 m2	1 ea	13,587 m2	$ 1,804.63 USD/M2	$ 24,519,625	
Security/Hardening Measures (3% of above)					$ 735,589	
US Design Contingency 15%				**In the report**	$ 4,386,667	
CN Design Contingency 20%					$ 5,051,043	
Approaches Total (rounded)						**$ 63,937,000**
Bridge Subtotal					**$ 656,624,000**	
CE 4 Construction Contingency 20%					$ 131,324,800	
Bridge Grand Total (rounded)					**$ 787,948,800**	**$ 790,000,000**



PARSONS

SUBJECT: **DRIC - Cable Stay Option Alternate A2**
14 Spans with Concrete I-Girders
JOB NO: 646294
MADE BY: OYM DATE: 10/2/2008 CHKD BY: DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	11,461	CY	$442.20	$5,068,231
		Excavation	8,596	CY	$8.00	$68,768
	7060020	Superstructure Concrete-Includes 7060022 (Form Finish & Cure)	5,706	CY	$799.00	$4,559,094
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	2,295,375	LB	$1.21	$2,777,404
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	1,169,730	LB	$1.10	$1,286,703
	7070073	Elastomeric Bearings (3" Thick)	858	SF	$185.00	$158,730
	7080101	Precast Concrete 1800 Beam - Furn	20,905	LF	$240.63	$5,030,439
	7080102	Precast Concrete 1800 Beam - Erect	20,905	LF	$30.00	$627,159
	7110001	Bridge Barrier Railing - Type 4	3,276	LF	$84.67	$277,396
	-	Expansion Joint Device (Modular 6")	441	LF	$123.00	$54,243
	-	Handrail Barrier - 42"	1,608	LF	$500.00	$804,050
	-	Misc (Lighting, Drainage, Signage, Stripping)	177,325	SF	$15.00	$2,659,876
	-	Drilled Pier (3' Diameter)	12,180	LF	$290.00	$3,532,200
	-	Drilled Pier (4' Diameter)	1,680	LF	$430.00	$722,400
		Environmental Remediation	14	each	$150,000.00	$2,100,000
		Bridge Area = (Deck Width * Bridge Length)	177325	SF		
		SUB-TOTAL BRIDGE STRUCTURE				**$29,726,692**
		MOBILIZATION (included in Michigan state historic prices)				
		DESIGN CONTINGENCY (taken at main bridge rollup)				
		TOTAL BRIDGE STRUCTURE				**$29,726,692**
		COST /SF BRIDGE	**$167.64**			
		COST /SM BRIDGE	**$1,804.63**			

PARSONS	**BRIDGE QUANTITY ESTIMATE SPREADSHEET**		
PROJECT:	**DRIC - Cable Stay Option Alternate A2**		
	14 Spans with Concrete I-Girders		
Date:	02-Oct-08	Date Checked:	
By:	OYM	Checked By:	

STEEL TO CONCRETE REINFORCEMENT RATIOS

Deck:	205.00	#/CY
Abutments:	100.00	#/CY
Single Column Piers >25':	210.00	#/CY

BRIDGE CHARACTERISTICS

			Raised Median Height:	0.00	inch
Skew Angle:	0.0000	deg	Raised Median Width:	0.00	feet
Bridge Length:	1608.099	feet	120 Day Haunch/Build up:	4.00	inch
Overhang:	3.875	feet (Avg.)	Beam Type (1-6, 0):	6	(Steel = 0)
Number of Spans:	14		Beam Top Flange Width:	47.25	inch
Number of Beams(Webs)/Span:	13		Beam (conc)/Web (steel) Depth:	70.88	inch
Spacing:	8.29	feet	Steel Girder Top Flange Width:	0.00	inch
Deck Width:	110.270	feet (Avg.)	Steel Girder Web Depth:	0.00	inch
Deck Slab Depth:	9.0	inch	End Bent Cap Length:	161.28	feet
No of Approach Slabs:	1		Avg. End Bent Cap Width:	4.00	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Height:	16.00	feet
Cross Slope:	0.02	ft/ft	End Bent Pedestal Length:	3.00	feet
# of Traffic Barrier Rails:	2		Avg End Bent Pedestal Height:	6.00	inch
Handrail Barrier:	1		Number of 3' Drilled Piers (per Abut):	12	(PRODUCTION)
Median Barrier:	0		Average Length of 3' Drilled Piers (Abut):	105.00	feet
Continuous Spans (Y/N):	N		Test Piles(End Bent):	0	(Total per bridge)
Total # of Expansion Joint Seals:	4		Test Pile Length:	0.00	feet
End Diaphragms (Y/N):	Y		Structural Steel/Deck Ratio:	0.00	psf
Intermediate Diaphragms (Y/N):	N				

PIER or BENT "A" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	6.00	feet	Total Cap Height:	15.00	feet
Average Column Height:	26.35	feet	Average Column Area:	90.00	Sq ft
Cap Length:	53.35	feet	Footing Length B:	18.00	feet
Cap Width:	7.00	feet	Footing Width D:	18.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	6.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. 3' Drilled Piers Length:	105.00	feet
# of 3' Drilled Piers per Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers w/3' DP:	25		Test Pile Length:	0.00	feet

PIER OR BENT "B" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet (Average)	Cap End Height:	5.00	feet
Column Width L (0 for Round):	6.00	feet	Total Cap Height:	15.00	feet
Average Column Height:	69.00	feet	Average Column Area:	90.00	Sq ft
Cap Length:	53.35	feet	Footing Length B:	18.00	feet
Cap Width:	7.00	feet	Footing Width D:	18.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	6.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. 4' Drilled Piers Length:	105.00	feet
# of 4' Drilled Piers per Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers w/4' DP:	4		Test Pile Length:	0.00	feet
			End Diaphragm:		

MISC. DATA

			Diaph. LINES (@ end of beam):	28	
Wingwall (plus):			Distance (CL-CL) of ext. beams:	99.48	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	76.00	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	12	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	875.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	571.84	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
3' Drilled Pier per Wingwall:	2		Cheek Wall ht:	9.00	feet
Estimated 3' Drilled Piers length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab :	368	Sq Yd			

(std length/cos skew)* bridge width)

DRIC - Cable Stay Option Alternate A2
14 Spans with Concrete I-Girders

QUANTITIES			
DECK CONCRETE			
Deck + Raised Med. + Haunch + Diaphragm			
Deck = Brdg Length * Deck Width * Slab Thickness:	4926		
Raised Med = Brdg Length * Med Width * Med Height:	0		
Haunch = (3 * Slope * Flange Width^2 + 2 * Flange Width * Build-Up) * Length /6 * Num Beams			
or Haunch = Avg Steel Top Flange * Build-Up * Length * Num Beams:	170		
Diaphragm ={[Diaph Lines* Thickness*[(Diaph/line*Beam Spa*bm ht)-(#bms*bm area+diaph/line*area excluded)]]}	610		
Total Deck:		5,706	CY
SUBSTRUCTURE CONCRETE			
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:			
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	74		
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	765		
Wing Cap = [#*L*W*H]	38		
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	4		
Wingwalls & Wingposts = {#*L*W*H}	114		
Cheek wall Total:(ht*wd*thickness*#walls)	2		
End Bent Total:	**996**		
Pier or Bent "A" & "B":			
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**	
Cap = Cap Length * Height * Width - Haunch Volume:	160	160	
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	5	5	
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	95	237	
Footings = B * D * T * Number of Footings:	78	78	
Pier or Bent Subtotal:	337	480	
*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**8425**		
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers):*	**1918**		
Total Substructure:		11,339	CY
BRIDGE FLOOR GROOVING			
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9		17,315	SY
BARRIER RAILING			
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	3276		
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0	3,276	LF
HANDRAIL BARRIER			
Bridge Length * No. Handrail Barriers:	1608	1,608	LF
Drilled Piers			
Number of Wingwall 3' Drilled Piers * Avg Length:	420		
Number of EB * Number of End Bent 3' Drilled Piers * Avg. Length:	1260		
Number of Pier 'A' - 3' Drilled Piers * Avg. Length:	10500		
Number of Pier 'B' - 4' Drilled Piers * Avg. Length:	1680		
Total Production 3' Drilled Piers:		12,180	LF
Total Production 4' Drilled Piers:		1,680	LF
ELASTOMERIC BEARINGS			
(((# Spans + # Exp. Jt. - 1)* # Beams) - 1/2 * # Beams) * 2' x 2' Pad	858	858	CF
EXPANSION JOINT			
Skew Length (deck width) * Total # expansion joint strips:		441	LF
BEAMS			
Brdg Length * Number of Beams:		20,905	LF
STRUCTURAL STEEL			
Deck Area * Struct Steel Ratio:		0	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)			
Superstructure:	1169730		
End Bents + Walls:	99600		
BENTS:	2172030	3,441,360	LB
APPROACH SLAB (2000 Std)			
Concrete Volume= Approach Slab Area * .333 CY per SQ YD		122	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)		23,745	LB

PARSONS

SUBJECT: **DRIC - Cable Stay Option Alternate B2**
8 Spans with Steel I-Girders

JOB NO: 646294

MADE BY: OYM DATE: 10/2/2008 CHKD BY: DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	7,451	CY	$442.20	$3,294,671
	7060020	Superstructure Concrete-Includes 7060022 (Form Finish & Cure)	5,182	CY	$799.00	$4,140,418
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	1,465,572	LB	$1.21	$1,773,343
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	1,062,310	LB	$1.10	$1,168,541
	7070007	Structural Steel, Plate - Furn/Fab (46 lb/sf)	7,812,665	LB/LS	$1.88	$14,687,810
	7070008	Structural Steel, Plate - Erect (46 lb/sf)	7,812,665	LB/LS	$0.18	$1,406,280
	7070073	Elastomeric Bearings (3" Thick)	96	SF	$185.00	$17,760
	7110001	Bridge Barrier Railing - Type 4	3,186	LF	$84.67	$269,718
	-	Expansion Joint Device (Modular 6")	326	LF	$500.00	$163,000
	-	Handrail Barrier - 42"	1,563	LF	$217.68	$340,181
	-	Lighting and Drainage ($5/SF)	169,841	SF	$5.00	$849,203
	-	Multirotational Bearing Assembly	72	EA	$4,000.00	$288,000
	-	Drilled Pier (3' Diameter)	2,520	LF	$290.00	$730,800
	-	Drilled Pier (4' Diameter)	5,880	LF	$430.00	$2,528,400
		Bridge Area = (Deck Width * Bridge Length)	169841	SF		
		SUB-TOTAL BRIDGE STRUCTURE				**$31,658,124**
		MOBILIZATION +5%				**$1,582,906**
		DESIGN CONTINGENCY +15%				**$4,748,719**
		TOTAL BRIDGE STRUCTURE				**$37,989,748**
		COST /SF BRIDGE	**$223.68**			

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

PROJECT: **DRIC - Cable Stay Option Alternate B2**
8 Spans with Steel I-Girders

Date: 02-Oct-08 Date Checked: ________
By: OYM Checked By:

STEEL TO CONCRETE REINFORCEMENT RATIOS

Deck:	205.00	#/CY
Abutments:	100.00	#/CY
Single Column Piers >25':	210.00	#/CY

BRIDGE CHARACTERISTICS

			Raised Median Height:	0.00	inch
Skew Angle:	0.0000	deg	Raised Median Width:	0.00	feet
Bridge Length:	1562.758	feet	120 Day Haunch/Build up:	4.00	inch
Overhang:	4.267	feet	Beam Type (1-6, 0):	0	(Steel = 0)
Number of Spans:	8		Beam Top Flange Width:	24.00	inch
Number of Beams(Webs)/Span:	12		Beam (conc)/Web (steel) Depth:	84.00	inch
Spacing:	10.00	feet	Steel Girder Top Flange Width:	24.00	inch
Deck Width:	108.680	feet (Avg.)	Steel Girder Web Depth:	84.00	inch
Deck Slab Depth:	9.0	inch	End Bent Cap Length:	136.97	feet
No of Approach Slabs:	1		Avg. End Bent Cap Width:	4.00	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Height:	16.00	feet
Cross Slope:	0.02	ft/ft	End Bent Pedestal Length:	3.00	feet
# of Traffic Barrier Rails:	2		Avg End Bent Pedestal Height:	6.00	inch
Handrail Barrier:	1		Number of 3' Drilled Piers (per Abut):	10	(PRODUCTION)
Median Barrier:	0		Average Length of Drilled Piers (Abut):	105.00	feet
Continuous Spans (Y/N):	Y		Test Piles(End Bent):	0	(Total per bridge)
Total # of Expansion Joint Seals:	3		Test Pile Length:	0.00	feet
End Diaphragms (Y/N):	N		Structural Steel/Deck Ratio:	46.00	psf
Intermediate Diaphragms (Y/N):	N				

PIER or BENT "A" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	6.00	feet	Total Cap Height:	15.00	feet
Average Column Height:	31.81	feet	Average Column Area:	90.00	Sq ft
Cap Length:	54.50	feet	Footing Length B:	21.00	feet
Cap Width:	7.00	feet	Footing Width D:	21.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	8.00	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of 4' Drilled Piers Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	10		Test Pile Length:	0.00	feet

PIER OR BENT "B" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet (Average)	Cap End Height:	5.00	feet
Column Width L (0 for Round):	7.50	feet	Total Cap Height:	15.00	feet
Average Column Height:	64.99	feet	Average Column Area:	112.50	Sq ft
Cap Length:	54.50	feet	Footing Length B:	21.00	feet
Cap Width:	7.00	feet	Footing Width D:	21.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	8.00	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of 4' Drilled Piers Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	4		Test Pile Length:	0.00	feet

MISC. DATA

			End Diaphragm:		
			Diaph. LINES (@ end of beam):	0	
Wingwall (plus):			Distance (CL-CL) of ext. beams:	0	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	113.78	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	0	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	0.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	0.00	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
Piles per Wingwall:	2.00		Cheek Wall ht:	9.00	feet
Estimated Pile length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab :	362	Sq Yd			
(std length/cos skew)* bridge width)					

DRIC - Cable Stay Option Alternate B2
8 Spans with Steel I-Girders

QUANTITIES

Item	"A"	"B"	Total	Unit
DECK CONCRETE				
Deck + Raised Med. + Haunch + Diaphragm				
Deck = Brdg Length * Deck Width * Slab Thickness:	4718			
Raised Med = Brdg Length * Med Width * Med Height:	0			
Haunch = (3 * Slope * Flange Width^2 + 2 * Flange Width * Build-Up) * Length /6 * Num Beams				
or Haunch = Avg Steel Top Flange * Build-Up * Length * Num Beams:	464			
Diaphragm ={[Diaph Lines* Thickness*[(Diaph/line*Beam Spa*bm ht)-(#bms*bm area+diaph/line*area excluded)]]}	0			
Total Deck:			5,182	CY
SUBSTRUCTURE CONCRETE				
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:				
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	76			
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	649			
Wing Cap = [#*L*W*H]	38			
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	4			
Wingwalls & Wingposts = {#*L*W*H}	114			
Cheek wall Total:(ht*wd*thickness*#walls)	2			
End Bent Total:	**883**			
Pier or Bent "A" & "B":				
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**		
Cap = Cap Length * Height * Width - Haunch Volume:	164	164		
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	5	5		
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	113	279		
Footings = B * D * T * Number of Footings:	131	131		
Pier or Bent Subtotal:	413	579		
*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**4130**			
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers):*	**2317**			
Total Substructure:			7,330	CY
BRIDGE FLOOR GROOVING				
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9			16,827	SY
BARRIER RAILING				
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	3186			
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0		3,186	LF
HANDRAIL BARRIER				
Bridge Length * No. Handrail Barriers:	1563		1,563	LF
Drilled Piers				
Number of Wingwall 3' Drilled Piers * Avg Length:	420			
Number of EB * Number of End Bent 3' Drilled Piers * Avg. Length:	2100			
Number of Pier 'A' - 4' Drilled Piers * Avg. Length:	4200			
Number of Pier 'B' - 4' Drilled Piers * Avg. Length:	1680			
Total Production 3' Drilled Piers:			2,520	LF
Total Production 4' Drilled Piers:			5,880	LF
MULTIROTATIONAL BEARINGS				
(# Spans - # Exp. Jt. + 1) * # Beams	72		72	EA
ELASTOMERIC BEARINGS				
((# Exp. Jt. - 1) * 2) * # Beams * 2 sqft Pad	96		96	SF
EXPANSION JOINT				
Skew Length (deck width) * Total # expansion joint strips:			326	LF
BEAMS				
Brdg Length * Number of Beams:			0	LF
STRUCTURAL STEEL				
Deck Area * Struct Steel Ratio:			7,812,665	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)				
Superstructure:	1062310			
End Bents + Walls:	88300			
BENTS:	1353870		2,504,480	LB
APPROACH SLAB (2000 Std)				
Concrete Volume= Approach Slab Area * .333 CY per SQ YD			121	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)			23,402	LB

PARSONS

SUBJECT: **DRIC - Cable Stay Option Alternate C2**
12 Spans with Segmental Girders

JOB NO: 646294

MADE BY: JDB DATE: 9/30/2008 CHKD BY: DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	10,032	CY	$442.20	$4,436,077
	-	Superstructure Concrete (Segmental Beams)	11,354	CY	$1,200.00	$13,624,757
	7060031	Expansion Joint Device	549	LF	$123.00	$67,527
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	2,004,635	LB	$1.21	$2,425,608
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	2,327,563	LB	$1.10	$2,560,319
	7070077	Elastomeric Bearings (4" Thick)	648	SF	$187.50	$121,500
	7110001	Bridge Barrier Railing - Type 4	3,334	LF	$84.67	$282,274
	-	Handrail Barrier	1,637	LF	$217.68	$356,322
	-	Lighting and Drainage ($5/SF)	179,667	SF	$5.00	$898,336
	-	Precast Segment Production	1	LS	$5,000,000.00	$5,000,000
	-	Post-Tensioning Steel (Longitudinal)	592,902	LB	$2.20	$1,304,383
	-	Post-Tensioning Steel (Transverse)	215,601	LB	$4.00	$862,402
	-	Drilled Shaft (4')	12,810	LF	$430.00	$5,508,300
		Bridge Area = (Deck Width * Bridge Length)	179667	SF		
		SUB-TOTAL BRIDGE STRUCTURE				**$37,447,807**
		MOBILIZATION +5%				**$1,872,390**
		DESIGN CONTINGENCY +15%				**$5,617,171**
		TOTAL BRIDGE STRUCTURE				**$44,937,368**
		COST /SF BRIDGE	**$250.11**			

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

PROJECT: **DRIC - Cable Stay Option Alternate C2**
12 Spans with Segmental Girders

Date: #REF! Date Checked: ______
By: #REF! Checked By:

STEEL TO CONCRETE REINFORCEMENT RATIOS

Deck:	205.00	#/CY
Abutments:	100.00	#/CY
Single Column Piers >25':	210.00	#/CY

BRIDGE CHARACTERISTICS

Skew Angle:	0.0000	deg	Raised Median Height:	0.00	inch
Bridge Length:	1636.909	feet	Raised Median Width:	0.00	feet
Overhang:	4.267	feet	120 Day Haunch/Build up:	4.00	inch
Number of Spans:	12		Beam Type (1-6, 0):	0	(Steel = 0)
Number of Beams(Webs)/Span:	2		Beam Top Flange Width:	0.00	inch
Spacing:	56.00	feet	Beam (conc)/Web (steel) Depth:	0.00	inch
Deck Width:	109.760	feet (Avg.)	Steel Girder Top Flange Width:	0.00	inch
Deck Slab Depth:	0.0	inch	Steel Girder Web Depth:	0.00	inch
No of Approach Slabs:	1		End Bent Cap Length:	158.20	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Width:	4.00	feet
Cross Slope:	0.02	ft/ft	Avg. End Bent Cap Height:	16.00	feet
# of Traffic Barrier Rails:	2		End Bent Pedestal Length:	3.00	feet
Handrail Barrier:	1		Avg End Bent Pedestal Height:	6.00	inch
Median Barrier:	0		Number of Drilled Piers (per Abut):	11	(PRODUCTION)
Continuous Spans (Y/N):	Y		Average Length of Drilled Piers (Abut):	105.00	feet
Total # of Expansion Joint Seals:	5		Test Piles(End Bent):	0	(Total per bridge)
End Diaphragms (Y/N):	N		Test Pile Length:	0.00	feet
Intermediate Diaphragms (Y/N):	N		Post-Tensioning (Long.)/Deck Ratio:	3.3	psf
			Post-Tensioning (Long.)/Deck Ratio:	1.2	psf
			Segmental Concrete Ratio:	1.625	cf/sf

PIER or BENT "A" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	3.00	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	0.00	feet
Column Width L (0 for Round):	8.00	feet	Total Cap Height:	12.00	feet
Average Column Height:	49.85	feet	Average Column Area:	120.00	Sq ft
Cap Length:	21.33	feet	Footing Length B:	18.00	feet
Cap Width:	8.00	feet	Footing Width D:	18.00	feet
Cap Height:	12.00	feet	Footing Thickness T:	6.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of Drilled Piers per Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	24		Test Pile Length:	0.00	feet

PIER OR BENT "B" DATA

Number of Cols (USE 1 for bent):	0		Cap Taper Length (1 Side):	0.00	feet
Column Width B or Equiv. Diam.:	0.00	feet (Average)	Cap End Height:	0.00	feet
Column Width L (0 for Round):	0.00	feet	Total Cap Height:	0.00	feet
Average Column Height:	0.00	feet	Average Column Area:	0.00	Sq ft
Cap Length:	0.00	feet	Footing Length B:	0.00	feet
Cap Width:	0.00	feet	Footing Width D:	0.00	feet
Cap Height:	0.00	feet	Footing Thickness T:	0.00	feet
Pedestal Length:	0.00	feet	Depth of Footing:	0.00	feet
Average Pedestal Height:	0.00	inch	Avg. Drilled Piers Length:	0.00	feet
# of Drilled Piers per Footing:	0		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	0		Test Pile Length:	0.00	feet

MISC. DATA

			End Diaphragm:		
Wingwall (plus):			Diaph. LINES (@ end of beam):	0	
			Distance (CL-CL) of ext. beams:	0	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	113.78	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	0	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	0.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	0.00	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
Drilled Piers per Wingwall:	2		Cheek Wall ht:	9.00	feet
Estimated Drilled pier length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab :	366	Sq Yd			
(std length/cos skew)* bridge width)					

DRIC - Cable Stay Option Alternate C2
12 Spans with Segmental Girders

SEGMENTAL BEAM CONCRETE			
(Segmental Concrete Ratio * Area of Deck) / 27 + 5%:	11354	11,354	CY
SUBSTRUCTURE CONCRETE			
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:			
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	6		
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	750		
Wing Cap = [#*L*W*H]	38		
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	1		
Wingwalls & Wingposts = {#*L*W*H}	114		
Cheek wall Total:(ht*wd*thickness*#walls)	2		
End Bent Total:	**910**		

Item	"A"	"B"
Pier or Bent "A" & "B":		
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**
Cap = Cap Length * Height * Width - Haunch Volume:	65	0
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	1	0
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	230	0
Footings = B * D * T * Number of Footings:	78	0
Pier or Bent Subtotal:	375	0

*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**9000**		
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers):*	**0**		
Total Substructure:		9,910	CY
BRIDGE FLOOR GROOVING			
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9		17,625	SY
BARRIER RAILING			
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	3334		
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0	3,334	LF
HANDRAIL BARRIER			
Bridge Length * No. Barriers	1637	1,637	LF
Drilled Piers			
Test Piles * Test Pile Length:	0	0	LF
# Wingwall Drilled Piers * Avg Length:	420		
2 EB * Number of End Bent Drilled Piers * Avg. Length:	2310		
(Number of Pier 'A' Drilled Piers - Test Piles) * Avg. Length:	10080		
(Number of Pier 'B' Drilled Piers - Test Piles) * Avg. Length:	0		
Total Production Drilled Piers:		12,810	LF
BEARINGS (Steel Girder Configurations)			
Cont. = (# Spans + 1) * # Beams; Simple = (# of Spans * 2) * # Num Beams: * 0.6 cf/brg)	0		
ELASTOMERIC BEARINGS			
(((# Spans - # Exp. Jt. +1) * 2 Bearings * 2 Beams)+(# Exp. Jt. * 2 Bearings * 2 Beams * 2 Ends))* 9SF	648	648	CF
EXPANSION JOINT			
Skew Length (deck width) * Total # expansion joint strips:		549	LF
BEAMS			
Brdg Length * Number of Beams:		0	LF
STRUCTURAL STEEL			
Deck Area * Struct Steel Ratio:		0	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)			
Superstructure:	2327563		
End Bents + Walls:	91000		
BENTS:	1890000	4,308,563	LB
APPROACH SLAB (2000 Std)			
Concrete Volume= Approach Slab Area * .333 CY per SQ YD		122	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)		23,635	LB

PARSONS

Calculation:

project DRIC | job no. 646294
by SC | date 22-Nov-08 | Subject Engineering Report Cost Estimate
chk | date

File: F:\646294_DRIC_Study\Final_Eng_Report_Sept_08\Submittals\Engineering Report\Engineering Report_FINAL_Nov 08\Bound Appendices\Appendix B - Cost Est\DRIC Master Cost Estimates r3.0 081120.xls

OPTION: PA Option 7 Caisson Option CROSSING: X-10(B)

Geometry

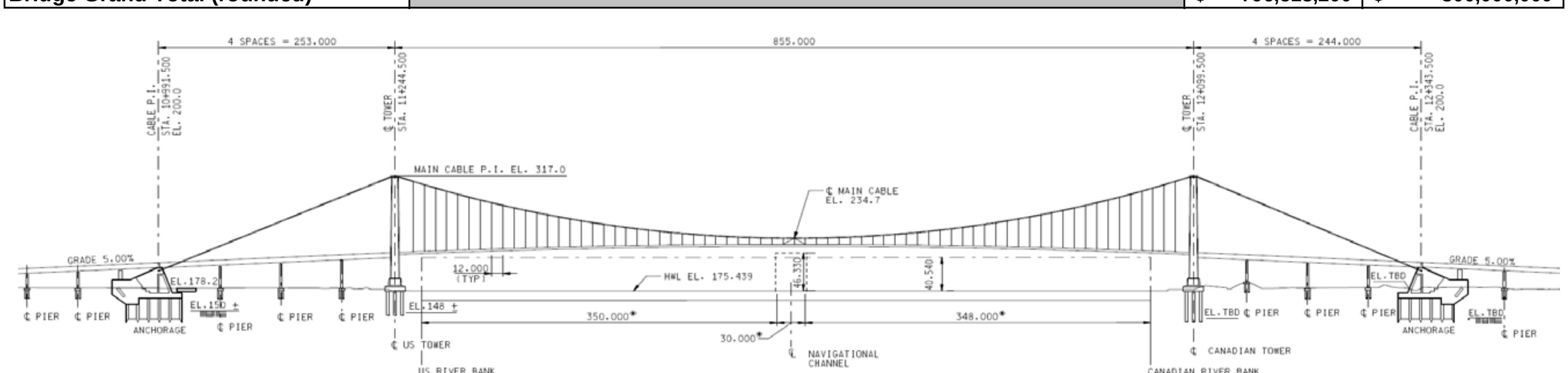
North Abutment:	STA	*10+423.85*	Main Span Lengtl	855	meters
North End of Suspended Spans:	STA	*11+244.50*	Suspended Span	855	meters
North Tower:	STA	*11+244.50*	US Approach Spa	821	meters
South Tower:	STA	*12+099.50*	CAN Approach S|	735	meters
South End of Suspended Spans:	STA	*12+099.50*	Back Stay Cable	*1100*	meters
South Abutment:	STA	*12+834.06*			
Length of Suspended Spans	855.00	meters	Cable-to-Cable W	*34.60*	meters
Length of Approaches	1,555.21	meters	Curb-to-Curb Wid	*31.52*	meters

SUSPENSION BRIDGE

REVISION PREPARED BY: Y.ARONSON / N.VOLDMAN

mate dated September 2,2008

Item	Revised Unit QTY	QTY Mult.	QTY Total	Unit Cost	Total	Total
Superstructure						
Deck						
Orthotropic Box Girder	371.85 kg/m^2	29,583 m^2	*11,000,400* kg	$ 12.02 USD/kg	$ 132,169,806	
Security/Hardening Measures (5% of above)					$ 6,608,490	
Deck Subtotal					*$ 138,778,296*	
Suspension System						
Main Cable Wire	2,208.00 kg/m/cable	3,000 m	*6,622,700* kg	$ 7.10 USD/kg	$ 47,047,661	
Spinning Equipment	1.00 LS	*1* ea	*1* ea	included USD/ea	included	
Catwalk	1.00 LS	*2* ea	*2* ea	$ 324,088.20 USD/	$ 648,176	
Wrapping wire	44.70 kg/m/cable	2,670 m	*119,000* kg	$ 4.20 USD/kg	$ 499,800	
Suspenders	83,050.00 kg/cable	*2* cable	*166,100* kg	$ 26.18 USD/kg	$ 4,349,162	
Castings	534,400.00 kg/br	*1* br	*534,400* kg	$ 32.68 USD/kg	$ 17,463,337	
Anchor Rods	159,000.00 kg/anchor	*2* anchor	*318,000* kg	$ 5.16 USD/kg	$ 1,640,880	
Security/Hardening Measures (5% of above)					$ 3,582,451	
Suspension System Subtotal					*$ 75,231,467*	
Misc. Appurtenances						
Overlay, Barriers and membrane	29,583.00 m^2/br	1 br	29,583 m^2	$ 300.00 USD/m2	$ 8,874,900	
Lighting and drainage	29,583.00 m^2/br	1 br	29,583 m^2	$ 100.00 USD/m2	$ 2,958,300	
Fiber Optic Cable		1 br	1,352 m	$ 147.60 USD/m	$ 199,555	
Fiber Optic Cable Node		1 br	2 ea	$ 110,000.00 USD/ea	$ 220,000	
Miscellaneous Items	29,583.00 m^2/br	1 br	29,583 m^2	$ 120.00 USD/m2	$ 3,549,960	
Misc. Appurtenances Subtotal					*$ 15,802,715*	
Superstructure Subtotal						***$ 229,812,000***
Substructure (cost Includes Excavation and Dewatering)						
Towers						
Concrete	40.84 m^3/m	*272* m for 2	*11,108* m^3	$ 3,349.16 USD/m3	$ 37,204,084	
Reinforcing Steel	178.00 kg/m^3	11,108 m^3	*1,977,309* kg	$ 2.91 USD/kg	$ 5,753,970	
Post Tensioning Steel	12.50 kg/m^3	11,108 m^3	*138,856* kg	$ 8.54 USD/kg	$ 1,185,830	
Stairs and Elevators for Access					$ 4,000,000	
Security/Hardening Measures (5% of above)					$ 2,407,194	
Towers Subtotal					*$ 50,551,079*	
Tower Foundations						
Footing Concrete	7,130.00 m^3/ea	*2* ea	*14,260* m^3	$ 588.67 USD/m3	$ 8,394,421	
Footing Reinforcing	80.74 kg/m^3	14,260 m^3	*1,151,300* kg	$ 2.91 USD/kg	$ 3,350,283	
Drilled Shaft Steel Casing (24mm)	1,953.00 kg/m	600 m	*1,171,800* kg	$ -	$ -	
Drilled Shaft (3mx96m + 1.5m rock socket)	24.00 shafts	*25* m/shaft	*600* m	$ -	$ -	
Drilled Shaft (3.0mx30m ø / tower),incl.casi	24 shafts	*213* m3/shaf	*5,123* m^3	$ 1,774.54 USD/m3	$ 9,090,094	
Rock Socket	24 shafts	*11* m3/shaf	*268* m^3	$ 3,408.13 USD/m3	$ 911,996	
Environmental Remediation			*1* ea	$ 5,000,000.00 USD/ea	$ 5,000,000	
Environmental Remediation			*1* ea	$ 450,000.00 USD/ea	$ 450,000	
Tower Foundations Subtotal					*$ 27,196,794*	
Anchorages						
Anchorage Concrete	35,552.00 m^3/ea	*2* ea	*71,104* m^3	$ 491.12 USD/m3	$ 34,920,276	
Anchorage Reinforcing	56.80 kg/m^3	71,104 m^3	*4,037,040* kg	$ 2.91 USD/kg	$ 11,748,959	
Caissons 10m x 60m x 1.2m thick-2 ea.per	20,697.00 m^3	*2* ea	*41,394* m^3	$ 1,301.49 USD/m3	$ 53,873,990	
Environmental Remediation			*1* ea	$ 1,000,000.00 USD/ea	$ 1,000,000	
Security/Hardening Measures (2% of above)					$ 2,030,865	
Anchorages Subtotal					*$ 103,574,090*	
Support Facilities						
Yard & barges		1 br	1 br	$ 7,000,000 ea	$ 7,000,000	
Subtotal					*$ 7,000,000*	
Substructure (cost Includes Excavation and Dewatering) Subtotal						***$ 188,322,000***
Quantities Subtotal (rounded)					***$ 418,134,000***	
General Conditions Bond and Insu 11%					$ 45,994,740	
GC's Overhead and Profit 10%					$ 46,412,874	
Design Contingency 10%					$ 51,054,161	
Main Bridge Total (rounded)						**$ 561,596,000**
Approaches						
US Approach Bridge	25,866.89 m2	1 ea	25,867 m2	$ 1,724.10 USD/M3	$ 44,597,169	
Security/Hardening Measures (3% of above)					$ 1,337,915	
CN Approach Bridge	23,153.33 m2	1 ea	23,153 m2	$ 1,724.10 USD/M3	$ 39,918,719	
Security/Hardening Measures (3% of above)					$ 1,197,562	
US Design Contingency 15%					**$ 6,890,263**	
CN Design Contingency 20%					$ 8,223,256	
Approaches Total (rounded)						**$ 102,165,000**
Bridge Subtotal					**$ 663,761,000**	
CE 7 Construction Contingency 20%					$ 132,752,200	
Bridge Grand Total (rounded)					**$ 796,513,200**	**$ 800,000,000**



PARSONS

SUBJECT: **DRIC - Suspension Option Alternate A1**
20 Spans with Concrete I-Girders

JOB NO: 646294

MADE BY: OYM DATE: 10/2/2008 CHKD BY: DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	18,118	CY	$442.20	$8,011,628
		Excavation	13,588	CY	$8.00	$108,706
	7060020	Superstructure Concrete-Includes 7060022 (Form Finish & Cure)	9,193	CY	$799.00	$7,345,207
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	3,693,217	LB	$1.21	$4,468,792
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	1,884,565	LB	$1.10	$2,073,022
	7070073	Elastomeric Bearings (3" Thick)	1,274	SF	$185.00	$235,690
	7080101	Precast Concrete 1800 Beam - Furn	25,512	LF	$240.63	$6,138,854
	7080102	Precast Concrete 1800 Beam - Erect	25,512	LF	$30.00	$765,348
	7110001	Bridge Barrier Railing - Type 4	5,386	LF	$84.67	$456,013
	-	Post-Tension Modified 1800 Beam - Furn	9,106	LF	$288.76	$2,629,406
	-	Post-Tension Modified 1800 Beam - Erect	9,106	LF	$36.00	$327,815
	-	Expansion Joint Device (Modular 6")	652	LF	$500.00	$326,000
	-	Post-Tensioning Steel	388,440	LB	$5.20	$2,019,888
	-	Handrail Barrier - 42"	2,663	LF	$217.68	$579,657
	-	Misc (Lighting, Drainage, Signage, Stripping)	289,456	SF	$15.00	$4,341,836
	-	Drilled Pier (3' Diameter)	8,820	LF	$290.00	$2,557,800
	-	Drilled Pier (4' Diameter)	9,240	LF	$430.00	$3,973,200
		Environmental Remediation	18	each	$150,000.00	$2,700,000
		Bridge Area = (Deck Width * Bridge Length)	289456	SF		
		SUB-TOTAL BRIDGE STRUCTURE				**$49,058,862**
		MOBILIZATION (included in Michigan state historic prices)				
		DESIGN CONTINGENCY (taken at main bridge rollup)				
		TOTAL BRIDGE STRUCTURE				**$49,058,862**
		COST /SF BRIDGE	**$160.16**			
			$ 1,724.10			

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

PROJECT: **DRIC - Suspension Option Alternate A1**
20 Spans with Concrete I-Girders

Date:	02-Oct-08	Date Checked:	
By:	OYM	Checked By:	

STEEL TO CONCRETE REINFORCEMENT RATIOS

Deck:	205.00	#/CY
Abutments:	100.00	#/CY
Single Column Piers >25':	210.00	#/CY

BRIDGE CHARACTERISTICS					
			Raised Median Height:	0.00	inch
Skew Angle:	0.0000	deg	Raised Median Width:	0.00	feet
Bridge Length:	2662.886	feet	120 Day Haunch/Build up:	4.00	inch
Overhang:	3.875	feet (Avg.)	Beam Type (1-6, 0):	6	(Steel = 0)
Number of Spans:	20		Beam Top Flange Width:	47.25	inch
Number of Beams(Webs)/Span:	13		Beam (conc)/Web (steel) Depth:	70.88	inch
Spacing:	8.29	feet (Avg.)	Steel Girder Top Flange Width:	0.00	inch
Deck Width:	108.700	feet (Avg.)	Steel Girder Web Depth:	0.00	inch
Deck Slab Depth:	9.0	inch	End Bent Cap Length:	161.29	feet
No of Approach Slabs:	1		Avg. End Bent Cap Width:	4.00	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Height:	16.00	feet
Cross Slope:	0.02	ft/ft	End Bent Pedestal Length:	3.00	feet
# of Traffic Barrier Rails:	2		Avg End Bent Pedestal Height:	6.00	inch
Handrail Barrier:	1		Number of 3' Drilled Piers (per Abut):	12	(PRODUCTION)
Median Barrier:	0		Average Length of Drilled Piers (Abut):	105.00	feet
Continuous Spans (Y/N):	Y		Test Piles:	0	(Total per bridge)
Total # of Expansion Joint Seals:	6		Test Pile Length:	0.00	feet
End Diaphragms (Y/N):	Y		Post-Tensioning /Deck Ratio:	5.20	psf
Intermediate Diaphragms (Y/N):	N				

PIER or BENT "A" DATA					
Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	6.00	feet	Total Cap Height:	15.00	feet
Average Column Height:	33.52	feet	Average Column Area:	90.00	Sq ft
Cap Length:	53.35	feet	Footing Length B:	18.00	feet
Cap Width:	7.00	feet	Footing Width D:	18.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	6.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. 3' & 4' Drilled Piers Length:	105.00	feet
# of 4' Drilled Piers per Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers W/3' DP:	17		Test Pile Length:	0.00	feet
# of Identical Piers W/4' DP:	12				

PIER OR BENT "B" DATA					
Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	7.50	feet (Avg.)	Total Cap Height:	15.00	feet
Average Column Height:	109.04	feet	Average Column Area:	112.50	Sq ft
Cap Length:	53.35	feet	Footing Length B:	25.00	feet
Cap Width:	7.00	feet	Footing Width D:	24.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	8.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. 4' Drilled Piers Length:	105.00	feet
# of 4' Drilled Piers per Footing:	5		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	8		Test Pile Length:	0.00	feet
			End Diaphragm:		

MISC. DATA					
			Diaph. LINES (@ end of beam):	40	
Wingwall (plus):			Distance (CL-CL) of ext. beams:	99.48	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	76.00	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	12	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	875.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	571.84	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
3' Drilled Piers per Wingwall:	2		Cheek Wall ht:	9.00	feet
Estimated 3' Drilled Piers length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab :	362	Sq Yd			
(std length/cos skew)* bridge width)					

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

DRIC - Suspension Option Alternate A1
20 Spans with Concrete I-Girders

QUANTITIES — Page: 2

DECK CONCRETE			
Deck + Raised Med. + Haunch + Diaphragm			
Deck = Brdg Length * Deck Width * Slab Thickness:	8040		
Raised Med = Brdg Length * Med Width * Med Height:	0		
Haunch = (3 * Slope * Flange Width^2 + 2 * Flange Width * Build-Up) * Length /6 * Num Beams			
or Haunch = Avg Steel Top Flange * Build-Up * Length * Num Beams:	281		
Diaphragm ={[Diaph Lines* Thickness*[(Diaph/line*Beam Spa*bm ht)-(#bms*bm area+diaph/line*area excluded)]]}	872		
Total Deck:		9,193	CY
SUBSTRUCTURE CONCRETE			
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:			
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	74		
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	765		
Wing Cap = [#*L*W*H]	38		
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	4		
Wingwalls & Wingposts = {#*L*W*H}	114		
Cheek wall Total:(ht*wd*thickness*#walls)	2		
End Bent Total:	**996**		
Pier or Bent "A" & "B":			
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**	
Cap = Cap Length * Height * Width - Haunch Volume:	160	160	
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	5	5	
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	118	463	
Footings = B * D * T * Number of Footings:	78	189	
Pier or Bent Subtotal:	361	816	
*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**10469**		
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers):*	**6532**		
Total Substructure:		17,997	CY
BRIDGE FLOOR GROOVING			
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9		28,658	SY
BARRIER RAILING			
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	5386		
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0	5,386	LF
HANDRAIL BARRIER			
Bridge Length * No. Handrail Barrier:	2663	2,663	LF
Drilled Piers			
Number of Wingwall 3' Drilled Piers * Avg Length:	420		
Number of EB * Number of End Bent 3' Drilled Piers * Avg. Length:	1260		
Number of Pier 'A'-3' Drilled Piers * Avg. Length:	7140		
Number of Pier 'A'-4' Drilled Piers * Avg. Length:	5040		
Number of Pier 'B' - 4' Drilled Piers * Avg. Length:	4200		
Total Production 3' Drilled Piers:		8,820	LF
Total Production 4' Drilled Piers:		9,240	LF
BEARINGS (Steel Girder Configurations)			
Cont. = (# Spans + 1) * # Beams; Simple = (# of Spans * 2) * # Num Beams: * 0.6 cf/brg)	0		
ELASTOMERIC BEARINGS			
(((# Spans + # Exp. Jt. - 1)* # Beams) - 1/2 * # Beams) * 2' x 2' Pad	1274	1274	SF
EXPANSION JOINT			
Skew Length (deck width) * Total # expansion joint strips:		652	LF
BEAMS			
Brdg Length (1962.43) * Number of 1800 Beams (13):		25,512	LF
Brdg Length (700.46)* Number of Post-Tension Beam (13):		9,106	LF
POST-TENSIONING STEEL			
Deck Area (700.46'*106.69') * Post-tension/Deck Ratio:		388,440	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)			
Superstructure:	1884565		
End Bents + Walls:	99600		
BENTS:	3570210	5,554,375	LB
APPROACH SLAB (2000 Std)			
Concrete Volume= Approach Slab Area * .333 CY per SQ YD		121	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)		23,407	LB

PARSONS

SUBJECT: **DRIC - Suspension Option Alternate B1**
13 Spans with Steel I-Girders

JOB NO: 646294

MADE BY: OYM DATE: 10/2/2008 CHKD BY: DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	12,651	CY	$442.20	$5,594,082
	7060020	Superstructure Concrete-Includes 7060022 (Form Finish & Cure)	8,543	CY	$799.00	$6,825,857
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	2,557,576	LB	$1.21	$3,094,667
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	1,751,315	LB	$1.10	$1,926,447
	7070007	Structural Steel, Plate - Furn/Fab (46 lb/sf)	12,970,250	LB/LS	$1.88	$24,384,070
	7070008	Structural Steel, Plate - Erect (46 lb/sf)	12,970,250	LB/LS	$0.18	$2,334,645
	7070073	Elastomeric Bearings (3" Thick)	132	SF	$185.00	$24,420
	7110001	Bridge Barrier Railing - Type 4	5,295	LF	$84.67	$448,335
	-	Expansion Joint Device (Modular 6")	431	LF	$500.00	$215,500
	-	Handrail Barrier - 42"	2,618	LF	$217.68	$569,787
	-	Lighting and Drainage ($5/SF)	281,962	SF	$5.00	$1,409,810
	-	Multirotational Bearing Assembly	110	EA	$4,000.00	$440,000
	-	Drilled Pier (4' Diameter)	11,970	LF	$430.00	$5,147,100
		Bridge Area = (Deck Width * Bridge Length)	281962	SF		
		SUB-TOTAL BRIDGE STRUCTURE				**$52,414,719**
		MOBILIZATION +5%				**$2,620,736**
		DESIGN CONTINGENCY +15%				**$7,862,208**
		TOTAL BRIDGE STRUCTURE				**$62,897,663**
		COST /SF BRIDGE	**$223.07**			

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

PROJECT: **DRIC - Suspension Option Alternate B1**
13 Spans with Steel I-Girders

Date: **02-Oct-08** Date Checked: ___________
By: **OYM** Checked By:

STEEL TO CONCRETE REINFORCEMENT RATIOS

Deck:	205.00	#/CY
Abutments:	100.00	#/CY
Single Column Piers >25':	210.00	#/CY

BRIDGE CHARACTERISTICS

			Raised Median Height:	0.00	inch
Skew Angle:	0.0000	deg	Raised Median Width:	0.00	feet
Bridge Length:	2617.545	feet	120 Day Haunch/Build up:	4.00	inch
Overhang:	3.875	feet (Avg.)	Beam Type (1-6, 0):	0	(Steel = 0)
Number of Spans:	13		Beam Top Flange Width:	24.00	inch
Number of Beams(Webs)/Span:	11		Beam (conc)/Web (steel) Depth:	84.00	inch
Spacing:	9.90	feet (Avg.)	Steel Girder Top Flange Width:	24.00	inch
Deck Width:	107.720	feet (Avg.)	Steel Girder Web Depth:	84.00	inch
Deck Slab Depth:	9.0	inch	End Bent Cap Length:	136.97	feet
No of Approach Slabs:	1		Avg. End Bent Cap Width:	4.00	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Height:	16.00	feet
Cross Slope:	0.02	ft/ft	End Bent Pedestal Length:	3.00	feet
# of Traffic Barrier Rails:	2		Avg End Bent Pedestal Height:	6.00	inch
Handrail Barrier:	1		Number of 3' Drilled Piers (per Abut):	10	(PRODUCTION)
Median Barrier:	0		Average Length of Drilled Piers (Abut):	105.00	feet
Continuous Spans (Y/N):	Y		Test Piles:	0	(Total per bridge)
Total # of Expansion Joint Seals:	4		Test Pile Length:	0.00	feet
End Diaphragms (Y/N):	N		Structural Steel/Deck Ratio:	46.00	psf
Intermediate Diaphragms (Y/N):	N				

PIER or BENT "A" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	6.00	feet	Total Cap Height:	15.00	feet
Average Column Height:	42.95	feet	Average Column Area:	90.00	Sq ft
Cap Length:	53.35	feet	Footing Length B:	25.00	feet
Cap Width:	7.00	feet	Footing Width D:	21.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	8.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of Drilled Piers per Footing:	5		Test Piles (Per Pier):	0	
# of Identical Piers:	14		Test Pile Length:	0.00	feet

PIER OR BENT "B" DATA

Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	18.35	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	5.00	feet
Column Width L (0 for Round):	7.50	feet (Average)	Total Cap Height:	15.00	feet
Average Column Height:	111.02	feet	Average Column Area:	112.50	Sq ft
Cap Length:	53.35	feet	Footing Length B:	25.00	feet
Cap Width:	7.00	feet	Footing Width D:	24.00	feet
Cap Height:	15.00	feet	Footing Thickness T:	8.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of Drilled Piers per Footing:	5		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	6		Test Pile Length:	0.00	feet

MISC. DATA

			End Diaphragm:		
			Diaph. LINES (@ end of beam):	0	
Wingwall (plus):			Distance (CL-CL) of ext. beams:	0	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	113.78	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	0	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	0.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	0.00	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
Drilled Piers per Wingwall:	2		Cheek Wall ht:	9.00	feet
Estimated Drilled Pier length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab : (std length/cos skew)* bridge width)	359	Sq Yd			

QUANTITIES

DECK CONCRETE			
Deck + Raised Med. + Haunch + Diaphragm			
Deck = Brdg Length * Deck Width * Slab Thickness:	7832		
Raised Med = Brdg Length * Med Width * Med Height:	0		
Haunch = (3 * Slope * Flange Width^2 + 2 * Flange Width * Build-Up) * Length /6 * Num Beams			
or Haunch = Avg Steel Top Flange * Build-Up * Length * Num Beams:	711		
Diaphragm ={[Diaph Lines* Thickness*[(Diaph/line*Beam Spa*bm ht)-(#bms*bm area+diaph/line*area excluded)]]}	0		
Total Deck:		8,543	CY
SUBSTRUCTURE CONCRETE			
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:			
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	76		
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	649		
Wing Cap = [#*L*W*H]	38		
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	4		
Wingwalls & Wingposts = {#*L*W*H}	114		
Cheek wall Total:(ht*wd*thickness*#walls)	2		
End Bent Total:	**883**		
Pier or Bent "A" & "B":			
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**	
Cap = Cap Length * Height * Width - Haunch Volume:	160	160	
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	4	4	
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	150	471	
Footings = B * D * T * Number of Footings:	165	189	
Pier or Bent Subtotal:	479	824	
*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**6706**		
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers):*	**4942**		
Total Substructure:		12,531	CY
BRIDGE FLOOR GROOVING			
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9		28,170	SY
BARRIER RAILING			
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	5295		
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0	5,295	LF
HANDRAIL BARRIER			
Bridge Length * No. Handrail Barriers	2618	2,618	CY
Drilled Piers			
Number of Wingwall Drilled Piers * Avg Length:	420		
Number of EB * Number of End Bent Drilled Piers * Avg. Length:	1050		
Number of Pier 'A' Drilled Piers * Avg. Length:	7350		
Number of Pier 'B' Drilled Piers * Avg. Length:	3150		
Total Production Drilled Piers:		11,970	LF
MULTIROTATIONAL BEARINGS			
(# Spans - # Exp. Jt. + 1) * # Beams	110	110	EA
ELASTOMERIC BEARINGS			
((# Exp. Jt. - 1) * 2) * # Beams * 2 sqft Pad	132	132	SF
EXPANSION JOINT			
Skew Length (deck width) * Total # expansion joint strips:		431	LF
BEAMS			
Brdg Length * Number of Beams:		0	LF
STRUCTURAL STEEL			
Deck Area * Struct Steel Ratio:		12,970,250	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)			
Superstructure:	1751315		
End Bents + Walls:	88300		
BENTS:	2446080	4,285,695	LB
APPROACH SLAB (2000 Std)			
Concrete Volume= Approach Slab Area * .333 CY per SQ YD		120	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)		23,196	LB

PARSONS	SUBJECT: DRIC - Suspension Option Alternate C1 18 Spans with Segmental Girders	JOB NO: 646294
	MADE BY: OYM DATE: 10/2/2008 CHKD BY:	DATE:

ITEM	Pay Item #	DESCRIPTION	QUANTITY	UNIT	UNIT COST	TOTAL COST
	7060010	Substructure Concrete (Includes 1 Approach Slab)	16,203	CY	$442.20	$7,165,110
	-	Superstructure Concrete (Segmental Beams)	18,439	CY	$1,200.00	$22,126,869
	7060031	Expansion Joint Device	650	LF	$500.00	$325,000
	7060035	Reinforcing Steel - Epoxy Coated (Substructure)	3,300,672	LB	$1.21	$3,993,813
	7060035	Reinforcing Steel - Epoxy Coated (Superstructure)	3,042,444	LB	$1.10	$3,346,688
	7070077	Elastomeric Bearings (4" Thick)	900	SF	$185.00	$166,500
	7110001	Bridge Barrier Railing - Type 4	5,443	LF	$84.67	$460,897
	-	Handrail Barrier - 42"	2,692	LF	$217.68	$585,934
	-	Lighting and Drainage ($5/SF)	291,783	SF	$5.00	$1,458,914
	-	Precast Segment Production	1	LS	$5,000,000.00	$5,000,000
	-	Post-Tensioning Steel (Longitudinal, 3.3 lb/sf))	962,884	LB	$2.20	$2,118,344
	-	Post-Tensioning Steel (Transverse, 1.2 lb/sf)	350,139	LB	$4.00	$1,400,558
	-	Drilled Pier (3' Diameter)	0	LF	$290.00	$0
	-	Drilled Pier (4' Diameter)	17,010	LF	$430.00	$7,314,300
		Bridge Area = (Deck Width * Bridge Length)	291783	SF		
		SUB-TOTAL BRIDGE STRUCTURE				$55,462,927
		MOBILIZATION +5%				$2,773,146
		DESIGN CONTINGENCY +15%				$8,319,439
		TOTAL BRIDGE STRUCTURE				$66,555,513
		COST /SF BRIDGE	$228.10			

PARSONS **BRIDGE QUANTITY ESTIMATE SPREADSHEET**

PROJECT: **DRIC - Suspension Option Alternate C1**
18 Spans with Segmental Girders

Date: #REF! Date Checked: ________
By: #REF! Checked By:

STEEL TO CONCRETE REINFORCEMENT RATIOS					
Segmental Box Beams:	165.00	#/CY			
Abutments:	100.00	#/CY			
Single Column Piers >25':	210.00	#/CY			
BRIDGE CHARACTERISTICS			Raised Median Height:	0.00	inch
Skew Angle:	0.0000	deg	Raised Median Width:	0.00	feet
Bridge Length:	2691.724	feet	120 Day Haunch/Build up:	0.00	inch
Overhang:	NA	feet	Beam Type (1-6, 0):	0	(Steel = 0)
Number of Spans:	18		Beam Top Flange Width:	0.00	inch
Number of Beams(Webs)/Span:	2		Beam (conc)/Web (steel) Depth:	0.00	inch
Spacing:	53.35	feet	Steel Girder Top Flange Width:	0.00	inch
Deck Width:	108.400	feet (Avg.)	Steel Girder Web Depth:	0.00	inch
Deck Slab Depth:	0.0	inch	End Bent Cap Length:	158.20	feet
No of Approach Slabs:	1		Avg. End Bent Cap Width:	4.00	feet
Riding Surface Width:	96.7848	feet	Avg. End Bent Cap Height:	16.00	feet
Cross Slope:	0.02	ft/ft	End Bent Pedestal Length:	3.00	feet
# of Traffic Barrier Rails:	2		Avg End Bent Pedestal Height:	6.00	inch
Handrail Barrier:	1		Number of Drilled Piers (per Abut):	11	(PRODUCTION)
Median Barrier:	0		Average Length of Drilled Piers (Abut):	105.00	feet
Continuous Spans (Y/N):	Y		Test Piles(End Bent):	0	(Total per bridge)
Total # of Expansion Joint Seals:	6		Test Pile Length:	0.00	feet
End Diaphragms (Y/N):	N		Post-Tensioning (Long.)/Deck Ratio:	3.3	psf
Intermediate Diaphragms (Y/N):	N		Post-Tensioning (Long.)/Deck Ratio:	1.2	psf
			Segmental Concrete Ratio:	1.625	cf/sf
PIER or BENT "A" DATA					
Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	3.00	feet
Column Width B or Equiv. Diam.:	15.00	feet	Cap End Height:	0.00	feet
Column Width L (0 for Round):	8.00	feet	Total Cap Height:	12.00	feet
Average Column Height:	48.00	feet	Average Column Area:	120.00	Sq ft
Cap Length:	21.33	feet	Footing Length B:	18.00	feet
Cap Width:	8.00	feet	Footing Width D:	18.00	feet
Cap Height:	12.00	feet	Footing Thickness T:	6.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of Drilled Piers per Footing:	4		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	24		Test Pile Length:	0.00	feet
PIER OR BENT "B" DATA			Strut Concrete:	65.00	CY
Number of Cols (USE 1 for bent):	1		Cap Taper Length (1 Side):	3.00	feet
Column Width B or Equiv. Diam.:	15.00	feet (Average)	Cap End Height:	0.00	feet
Column Width L (0 for Round):	8.00	feet	Total Cap Height:	12.00	feet
Average Column Height:	113.00	feet	Average Column Area:	120.00	Sq ft
Cap Length:	21.33	feet	Footing Length B:	25.00	feet
Cap Width:	8.00	feet	Footing Width D:	24.00	feet
Cap Height:	12.00	feet	Footing Thickness T:	8.50	feet
Pedestal Length:	3.00	feet	Depth of Footing:	2.00	feet
Average Pedestal Height:	6.00	inch	Avg. Drilled Piers Length:	105.00	feet
# of Drilled Piers per Footing:	5		Test Piles (Per Pier):	0	
# of Identical Piers or Bents:	8		Test Pile Length:	0.00	feet
			End Diaphragm:		
MISC. DATA			Diaph. LINES (@ end of beam):	0	
Wingwall (plus):			Distance (CL-CL) of ext. beams:	0	feet
Total No. Wingwalls:	2		Ht of Diaph incl bot bm flange+build-up:	113.78	inch
Wingwall ht:	16.00	feet	Diaphragms per LINE:	0	
Wingwall Length:	32.00	feet	Diaphragm thickness:	12.00	inch
Wingwall thickness:	3.00	feet	Area of Beam to exclude:	0.00	Sq in
Wingwall Cap length:	32.00	feet	Rectangular Area to be exclude:	0.00	Sq in
Wingwall Cap width:	4.00	feet	Intermediate Diapgragm:		
Wingwall Cap thickness:	4.00	feet	Diapgragm thickness:	0.00	inch
Wing end post length:	0.00	feet	Diaph. LINES (@ mid span):	0	
Wing end post width:	0.00	feet			
Wing end post ht:	0.00	feet	Total No. Cheekwalls:	2	
Drilled Piers per Wingwall:	2		Cheek Wall ht:	9.00	feet
Estimated Drilled Pier length:	105.00	feet	Cheek Wall width:	4.00	feet
Approach Slab Standard Length:	30.00	feet	Cheek Wall thickness:	9.00	inches
Plan Area of 1 Approach Slab : (std length/cos skew)* bridge width)	361	Sq Yd			

DRIC - Suspension Option Alternate C1
18 Spans with Segmental Girders

SEGMENTAL BEAM CONCRETE			
(Segmental Concrete Ratio * Area of Deck) / 27 + 5%:	18439	18,439	CY
SUBSTRUCTURE CONCRETE			
End Bent Volume = Back Wall + Cap + Pedestal + Wingwall + Cheekwall:			
Backwall = 2 * Deck Width/cos(skew) * Wall Height * Wall Thickness:	6		
Cap = 2 * Deck Width/cos(skew) * Cap Height * Cap Width:	750		
Wing Cap = [#*L*W*H]	38		
Pedestal = 2 * [Num Beams * Ped. Height * Ped. Length *(Cap Width-1)]:	1		
Wingwalls & Wingposts = {#*L*W*H}	114		
Cheek wall Total:(ht*wd*thickness*#walls)	2		
End Bent Total:	**910**		
Pier or Bent "A" & "B":			
Volume = Cap + Pedestals + Columns + Footings:	**"A"**	**"B"**	
Cap = Cap Length * Height * Width - Haunch Volume:	65	65	
Pedestals = Num Beams * Avg. Ped. Height * Cap Width * Ped. Length	1	1	
Columns = (Avg. Col Height + Footing Depth) * Avg. Col. Area * Number of Cols. :	222	511	
Footings = B * D * T * Number of Footings:	78	189	
Pier or Bent Subtotal:	366	766	
*Pier or Bent "A" Total (Subtotal * Number of Identical Piers):*	**8784**		
*Pier or Bent "B" Total (Subtotal * Number of Identical Piers + Strut Conc. * # Piers / 2):*	**6389**		
Total Substructure:		16,083	CY
BRIDGE FLOOR GROOVING			
Riding Surface * (Bridge Length + (# App Slabs * 2' App Slab Length))/ 9		28,968	SY
BARRIER RAILING			
Traffic = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	5443		
Median = (# Barriers * Bridge Length) + (# Barriers * # App Slabs * (App Slab Length/cos(skew)))	0	5,443	LF
HANDRAIL BARRIER			
Bridge Length * No. Barriers	2692	2,692	CY
Drilled Piers			
Test Piles * Test Pile Length:	0	0	LF
# Wingwall Drilled Piers * Avg Length:	420		
2 EB * Number of End Bent Drilled Piers * Avg. Length:	2310		
(Number of Pier 'A' Drilled Piers - Test Piles) * Avg. Length:	10080		
(Number of Pier 'B' Drilled Piers - Test Piles) * Avg. Length:	4200		
Total Production Drilled Piers:		17,010	LF
BEARINGS (Steel Girder Configurations)			
Cont. = (# Spans + 1) * # Beams; Simple = (# of Spans * 2) * # Num Beams: * 0.6 cf/brg)	0		
ELASTOMERIC BEARINGS			
(((# Spans - # Exp. Jt. +1) * 2 Bearings * 2 Beams)+(# Exp. Jt. * 2 Bearings * 2 Beams * 2 Ends))* 9SF	900	900	SF
EXPANSION JOINT			
Skew Length (deck width) * Total # expansion joint strips:		650	LF
BEAMS			
Brdg Length * Number of Beams:		0	LF
STRUCTURAL STEEL			
Deck Area * Struct Steel Ratio:		0	LB
STEEL REINFORCEMENT (based on Steel/Concrete Reinf Ratio)			
Superstructure:	3042444		
End Bents + Walls:	91000		
BENTS:	3186330	6,319,774	LB
APPROACH SLAB (2000 Std)			
Concrete Volume= Approach Slab Area * .333 CY per SQ YD		120	CY
Reinforcing Steel = Approach Slab Area * 64.6 lb per SQ YD (194 lbs/CY)		23,342	LB

URS

TS Option 4 ***Cable Stayed*** ***Crossing: X10(B)***

LIFE CYCLE ANALYSIS

Item	Units	Unit Cost	Quantity	Item Cost	(yrs) Frequency	Number of Occurences	Net Present Cost	Net Present Cost	Net Present Cost
Discount Rate							3.00%	5.00%	7.00%
Initial Construction							$442,000,000	$442,000,000	$442,000,000
Visual Inspection and Report	LS	$60,000.00	1	$60,000	2	59	$955,110	$583,516	$413,937
In-Depth Inspection and report	LS	$300,000.00	1	$300,000	2	59	$4,775,548	$2,917,581	$2,069,687
Replace Bearings	EA	$10,000.00	20	$200,000	25	4	$171,823	$82,996	$45,100
Replace Expansion Joints	m	$80,000.00	63	$5,040,000	25	4	$4,329,933	$2,091,510	$1,136,517
Replace Stay Cables	Kg	$13.20	3,322,107	$43,851,812	75	1	$3,943,758	$1,030,095	$262,871
Replace Suspension Bridge Suspenders	Ea	$25,000.00	0	$0	50	1	$0	$0	$0
General Concrete/Struct. Steel Repairs	LS	$22,100,000.00	1	$22,100,000	50	1	$5,706,077	$2,041,926	$769,797
Overlay	Sq. m	$75.00	46,650	$3,498,720	15	7	$5,989,030	$3,223,452	$1,987,369
Tower Access Maintanence	LS	$50,000.00	2	$100,000	25	4	$85,911	$41,498	$22,550
Aviation Warning Lighting System	LS	$5,500.00	2	$11,000	20	5	$12,936	$6,603	$3,829
Roadway/Aesthetic Lighting	LS	$500,000.00	1	$500,000	35	2	$253,309	$108,967	$51,507
Drainage System	LS	$120,000.00	1	$120,000	25	4	$103,094	$49,798	$27,060
Railings/Barriers	m	$300.00	2,960	$888,000	25	4	$762,893	$368,504	$200,244
Paint Steel	Sq. m	$65.00	35,000	$2,275,000	20	5	$2,675,345	$1,365,574	$791,856

Economic Life (yrs) 120	TOTAL LIFE CYCLE COST =	**$472,000,000**	**$456,000,000**	**$450,000,000**
	CABLE-STAYED DECK AREA =	**46,650 m2**	**46,650 m2**	**46,650 m2**
	UNIT COST =	**$10117.99/m2**	**$9775.00/m2**	**$9646.38/m2**
	UNIT COST =	**$940.47/sf**	**$908.59/sf**	**$896.64/sf**

PARSONS

TS Option 7 ***Suspension*** ***Crossing: X10(B)***

LIFE CYCLE ANALYSIS

Item	Units	Unit Cost	Quantity	Item Cost	(yrs) Frequency	Number of Occurences	Net Present Cost	Net Present Cost	Net Present Cost
Discount Rate							3.00%	5.00%	7.00%
Initial Construction							$487,000,000	$487,000,000	$487,000,000
Visual Inspection and Report	LS	$60,000.00	1	$60,000	2	59	$955,110	$583,516	$413,937
In-Depth Inspection and report	LS	$300,000.00	1	$300,000	2	59	$4,775,548	$2,917,581	$2,069,687
Replace Bearings	EA	$25,000.00	6	$150,000	25	4	$128,867	$62,247	$33,825
Replace Expansion Joints	m	$80,000.00	63	$5,040,000	25	4	$4,329,933	$2,091,510	$1,136,517
Rewrap Suspension Bridge Cables	m	$2,000.00	2,860	$5,720,000	75	1	$514,421	$134,365	$34,289
Replace Suspension Bridge Suspenders	Ea	$25,000.00	138	$3,450,000	50	1	$890,768	$318,762	$120,172
General Concrete/Struct. Steel Repairs	LS	$24,350,000.00	1	$24,350,000	50	1	$6,287,012	$2,249,815	$848,170
Overlay	Sq. m	$75.00	42,615	$3,196,128	15	7	$5,471,060	$2,944,667	$1,815,489
Tower Access Maintanence	LS	$50,000.00	2	$100,000	25	4	$85,911	$41,498	$22,550
Aviation Warning Lighting System	LS	$5,500.00	2	$11,000	20	5	$12,936	$6,603	$3,829
Roadway/Aesthetic Lighting	LS	$500,000.00	1	$500,000	35	2	$253,309	$108,967	$51,507
Drainage System	LS	$120,000.00	1	$120,000	25	4	$103,094	$49,798	$27,060
Railings/Barriers	m	$300.00	2,960	$888,000	25	4	$762,893	$368,504	$200,244
Paint Steel	Sq. m	$65.00	36,000	$2,340,000	20	5	$2,751,783	$1,404,590	$814,481

Economic Life (yrs) 120	TOTAL LIFE CYCLE COST =	**$514,000,000**	**$500,000,000**	**$495,000,000**
	CABLE-STAYED DECK AREA =	**42,615 m2**	**42,615 m2**	**42,615 m2**
	UNIT COST =	**$12061.47/m2**	**$11732.95/m2**	**$11615.62/m2**
	UNIT COST =	**$1121.12/sf**	**$1090.58/sf**	**$1079.68/sf**