

# DETROIT RIVER INTERNATIONAL CROSSING

## BRIDGE TYPE STUDY REPORT

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## **1. Executive Summary**

### **1.1. Project Background & Report Scope**

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 international crossing, that met the project's purpose and need.

Through a comprehensive technical evaluation process, with input from the public, an Area of Continued Analysis (**Figure 4**) incorporating the two crossing corridors, X10 and X11, was identified for the development of Practical Alternatives. The U.S. and Canadian study teams developed specific bridge alignments coordinated with U.S. and Canadian plaza options and physical project constraints.

The scope of this report is to address the main bridge crossing the Detroit River, the options developed and considered, and to evaluate the technical merits of those options. The Practical Alternative design process will consist of two phases; Phase 1, is the structural Type Study (TS Phase); and, Phase 2, is the Conceptual Design (CD Phase). The TS Phase focuses on the main structure over the Detroit River, but includes approach structures in the comparative cost estimates such that total crossing costs may be compared. Other project components, such as the plaza, connecting roadways, and interchanges will be evaluated separately and are not addressed in this report.

### **1.2. Crossings Considered**

Based on the locations of the toll and inspection plaza options under consideration, geotechnical considerations, as well as the avoidance of major industries and cultural properties, three horizontal alignments were developed, X10(A), X10(B) and X11(C), as shown in **Figure 1**.



Figure 1. Crossings Considered.

**1.3. Engineering**

This report details the structural type study of the main bridge for the three crossing alignments. This study focuses on the main river bridge since bridge approaches will not significantly affect the ranking of crossing options within a particular corridor at this level of detail.

### 1.3.1. Geology

Bedrock in the project area generally consists of sedimentary rock, such as limestone and dolomite, interspersed with salt layers. The upper rock is Dundee Limestone with a weathered surface that is generally level. A hardpan layer, around 2.5 to 3 m (8 to 10 ft) thick, of highly over-consolidated glacial till overlies the bedrock formation. Given the glacial origins of the hardpan layer, occasional cobbles and large boulders are typically present in this layer. The overburden in the area generally consists of clay from 27 to 30 m (90 to 100 feet) thick which frequently contains intermittent sand and gravel layers.

These geological conditions are favorable for common large bridge foundation types such as drilled shafts and sunken caissons. At this time, drilled shafts are expected to provide the most efficient way to carry the vertical foundation loads.

As noted in other project reports [1], the history of brine well mining as well as the known sinkhole on the Canadian side, are of significant concern with regard to the location of the bridge and its foundations. The bridge alignments were developed in order to minimize the risk from known or suspected brine well locations. In addition, an extensive geophysical subsurface investigation program is being undertaken to ensure that the bridge foundations are founded on competent bedrock.

### 1.3.2. Cross Section

The cross section used for this study was developed in the *River Crossing Bridge Cross Section* Technical Memo [2]. It is subject to refinement based on on-going work with the Partnership.

The cross section maintains six (6) 3.75m (12'-4") travel lanes, and a 3.0m (9'-10") right shoulder, TL-4 exterior railing, a single 1.6m (5'-3"), sidewalk for pedestrian use only interior to the suspension system, bicycle traffic will be allowed to use each right shoulder – which will be striped for one way bicycle traffic, as shown in Figure 2 below.

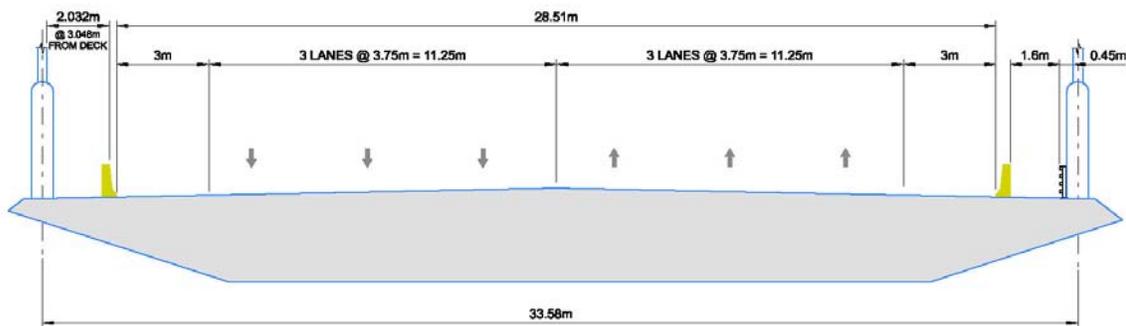
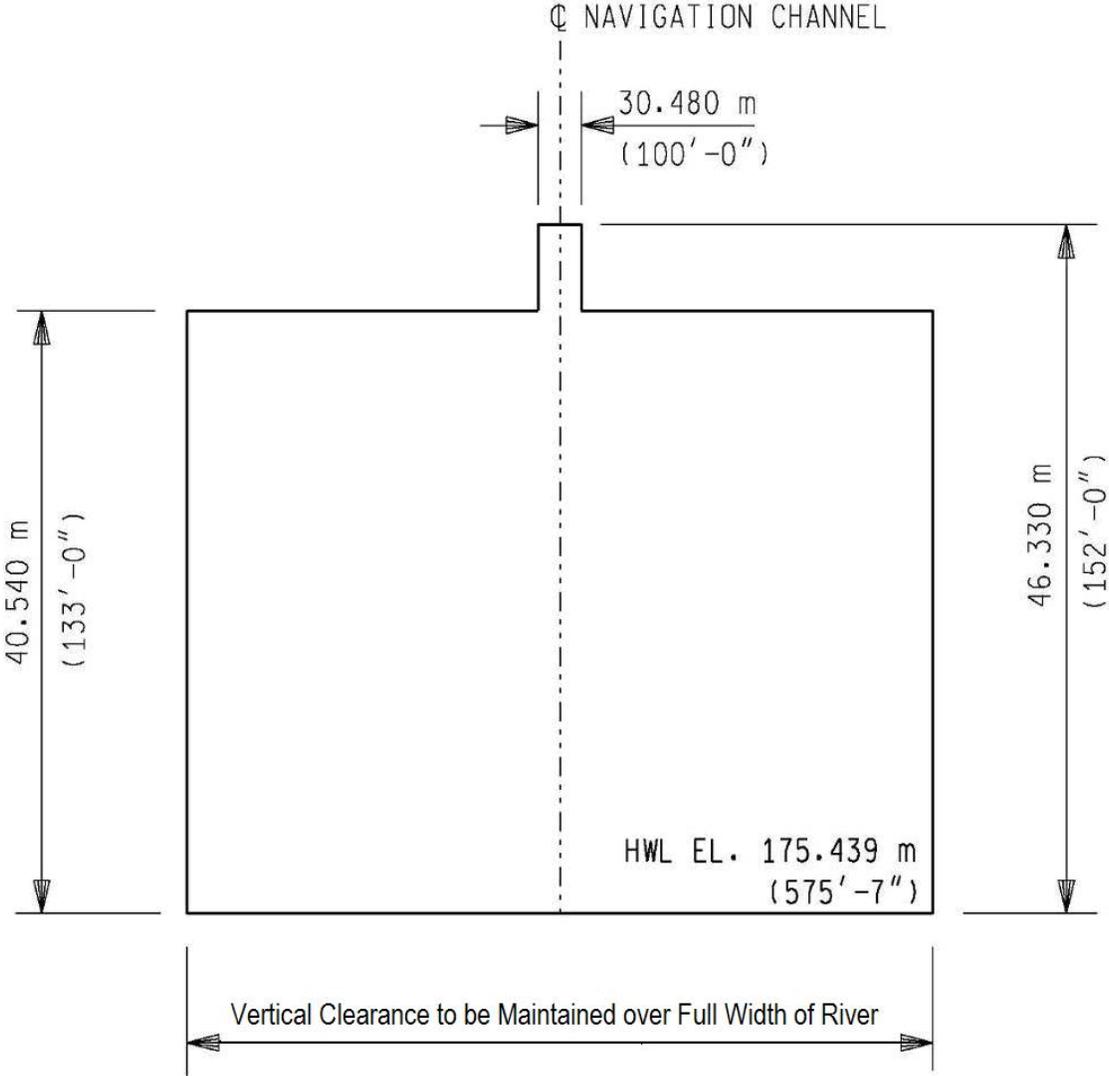


Figure 2. Cross Section.

### 1.3.3. Design Criteria

For the main structure crossing the river, the design codes of both U.S. and Canada apply. The project will be developed using the International System of Units (SI) (metric).

At this stage due to the concept level of design of the project the most significant design criteria is the navigation envelope shown in Figure 3. The navigation envelope is based on consultations with the U.S. Coast Guard and Transport Canada, as well as shipping industry representatives and is intended to provide at a minimum a navigation clearance the same as at the Ambassador Bridge.



Note: Vertical Scale x 10

Note: All dimensions shown perpendicular to the proposed channel.

Figure 3. Navigation Envelope.

### **1.3.4. Bridge Types Considered**

In the vicinity of corridors X10 and X11 the Detroit River varies in width from 570 m to 790 m. Currently there is major commercial shipping on the Detroit River as well as many shoreline industries in the project area which receive delivery of materials via ship. Therefore, it is necessary to provide a navigational envelope of adequate size so as not to restrict marine traffic. To achieve this, the bridge must span the entire river with a single clear span or place a single river pier adjacent to the navigation envelope. A river pier may provide adequate clearances while reducing the span length and associated cost of the crossing.

The resulting span range of 600 m to 1,300 m (1,968 ft to 4,265 ft) has only two viable bridge types. Suspension bridges can be utilized throughout this entire range, while cable-stayed bridges have a practical upper span limit of about 1000 m. (See **Table 6** and **Table 7** for lists of the world's longest Suspension and Cable-Stayed Bridges.)

## **1.4. Bridge Evaluation**

### **1.4.1. Methodolgy and Criteria**

The evaluation process will consist of two phases; Phase 1 is the structural Type Study (TS Phase); and, Phase 2 is the Conceptual Design (CD Phase). Other project components, the plaza, connecting roadways, and interchanges will be evaluated separately.

The evaluation process consists of scoring of screening criteria by competent bridge professionals from Parsons and URS with incorporation of Partnership input at appropriate times. At the conclusion of each development phase, the consultant team will evaluate each of the bridge options using the screening criteria in **Table 1**. This process will result in a consensus of options to retain for further study.

Below is a summary of the screening criteria to be used to evaluate the alternatives in the Type Study and Conceptual Design Phases. Each screening criterion will be evaluated using several performance factors, described in more detail in **Section 16.2**.

*Table 1. Screening Criteria.*

Screening Criteria	Practical Alternative Phase	
	Type Study	Conceptual Design
Initial Cost	X	X
Life-Cycle Cost	n/a	X
Constructability	X	X
Aesthetics	n/a	X
Safety and Security	X	X

### 1.4.2. Cost

Table 2 shows the cost range of the Type Study Options in 2006 US dollars. These costs were developed using a comparative estimation methodology with limited engineering as described in more detail in Section 13.1. Estimates include design and construction contingencies, but do not include a management contingency, engineering costs, property acquisition or environmental remediation costs.

*Table 2. Construction Cost Estimates.*

Crossing	Type Study Option	Bridge Type	River Pier	Construction Cost Estimate – 2006 US\$ (000,000's)
X10(A)	Option 1	Susp.	N	770 - 920
	Option 2	Susp.	Y	680 - 810
	Option 3	CS	Y	620 - 740
X10(B)	Option 4	CS	N	430 - 510
	Option 5	CS	Y	370 - 440
	Option 6	Susp.	N	480 - 550
	Option 7	Susp.	N	470 - 540
	Option 8	Susp.	Y	420 - 490
X11(C)	Option 9	CS	N	450 - 530
	Option 10	Susp.	N	500 - 580
	Option 11	Susp.	N	520 - 600

### 1.4.3. Constructability

All TS Options are within the limits of existing structures and are therefore considered constructible, the X10(A) structures present some challenges or risk due to their length. Table 1 presents the expected construction durations for the Options.

*Table 3. Construction Durations.*

Type Study Option	Duration (months)
<b>Crossing X10(A)</b>	
Option 1	62
Option 2	55
Option 3	57
<b>Crossing X10(B)</b>	
Option 4	52
Option 5	43
Option 6	49
Option 7	49
Options8	43
<b>Crossing X11(C)</b>	
Options 9	42
Option 10	51
Option 11	43

#### **1.4.4. Safety and Security**

Generally speaking all structure types under consideration may be adequately designed to mitigate safety and security risks. Presently there are no safety or security issues, either natural or man-made, that differentiate significantly between the structure types being considered on each alignment.

There are two issues which merit some concern. One is the risk presented due to vulnerability to ship impact for those options with piers in the water. This risk is mitigated by the design of pier protection in accordance with accepted standards, which is presented in more detail in **Section 3.4**. Second, is the risk presented by the potential operation of the Sterling Fuels facility which is being examined in greater depth? Some measures may be necessary to mitigate this concern.

### **1.5. Conclusions**

Cost, cost risk, schedule duration, schedule risk, and vulnerability to ship impact were considered to be the major differentiators between options on each crossing alignment after an evaluation of the data presented in this report. Some evaluation factors did not vary from option to option along an alignment. **Section 16.2** presents all of the evaluation factors.

In order to maintain a consistent approach to the development and evaluation of bridge options throughout the Practical Alternative phase of the study it is recommended that two options be retained at each crossing alignment. While it is recommended, from a technical perspective, that these options be retained for further study, as discussed earlier, it is recognized that Crossing X10(A) is not preferred from a bridge engineering perspective. Therefore,

consideration will be given to postponing the advancement of the conceptual design for crossing X10(A) until preliminary results are obtained from the geotechnical investigation program and any other relevant project EA/EIS studies.

The final recommended options, presented in this report and based on data received to date, clear span the river and do not have piers in the water. Although options with piers in the water were on the order of \$60 to \$110 million less costly than equivalent structure types without marine piers, input from both the U.S. and Canadian Lake Carriers Association, River pilots, and the U.S. Coast Guard made strong objection to piers in the river citing navigation issues related to docking on both the U.S. and Canadian shores and navigation entering and exiting the River Rouge. These objections were considered compelling and led to recommendation at all locations to clear span the river. Table 4 presents the final recommended options for each alignment.

*Table 4. Options Recommended for Further Study*

Type Study Option Elevation	Type Study Option
<b>X10(A)</b>	
	Option 1
<b>X10(B)</b>	
	Option 4
	Option 7
<b>X11(C)</b>	
	Option 9
	Option 10

## **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 crossing structure.

Through a comprehensive technical evaluation process, with input from the public an Area of Continued Analysis (**Figure 4**) incorporating, the two crossing corridors X10 and X11, was identified for the development of Practical Alternatives. The bridge Options are being advanced through a two-step process; Phase 1, is the structural Type Study (TS Phase); and, Phase 2, is the Conceptual Design (CD Phase). This report documents the development of the eleven (11) Practical Alternatives advanced through the Type Study Phase.

### **2.2. Report Scope**

The scope of this Type Study Report is to document the development process for the main bridge crossing the Detroit River, the options developed and considered, to evaluate the technical merits of those options, and to recommend alternatives for further development during the Conceptual Design Phase. A later report will document the CD Phase.

The TS Phase considers the entire crossing structure (i.e., main span and approach spans) but will focus on the main structure over the Detroit River. Other project components, such as the plaza, connecting roadways, and interchanges will be evaluated separately and are not addressed in this report.

In coordination with this technical process, a comprehensive Context Sensitive Solutions (CSS) process is being undertaken with the project stakeholders. The CSS process and results will be the subject of other reports.

The goal of the Type Study design process is to identify and recommend the most attractive options to be advanced during the Conceptual Design phase. It is noted, however, that the highest rated bridge may not be the most favorable option, as the evaluation of other project components will factor into the selection of a Preferred Alternative. However, the evaluation will yield a preferred bridge option for each crossing alignment.

### **2.3. Crossing Locations**

Two crossing corridors were identified in the Illustrative Alternative phase, X10 and X11, 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 4**, 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 X10 and X11 river crossing corridors were reexamined.

Based on the avoidance of major industries and cultural properties such as Brighton Beach Power Station, Fort Wayne, and Mistersky Power Plant, two horizontal alignments were developed, X10(B) and X11(C), **Figure 1**. A third horizontal alignment – X10(A) – was developed to avoid the area around a known sinkhole from historical brine mining in Canada. The alignment starts near the location of X10(B) in the U.S. and lands in Canada south west of Brighton Beach Power Station.



Figure 4. Area of Continued Analysis.

## 2.4. Bridge Types

In the vicinity of corridors X10 and X11 the Detroit River varies in width from 570m to 790m. Currently there is major commercial shipping on the Detroit River as well as many shoreline industries in the project area which 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. To achieve this, the bridge must span the entire river with a single clear span (i.e., both main towers are on the shore), or a single river pier adjacent to the navigation envelope may also provide adequate clearances while minimizing the span length and associated cost of the crossing. Navigation requirements are addressed in **Section 3.3**. Also, given the skew of the horizontal alignments necessary to avoid physical constraints the bridge span lengths are in excess of 600m. At this length the only practical bridge types are cable-stayed and suspension bridges.

The crossing locations for the Detroit River that are being considered are described in **Section 2.3** of this report. They include three horizontal alignments that were developed in consideration of project constraints. The alignments cross the river at a skew angle of 49 degrees for alignment X10(A) (skew angle measured from a line perpendicular to the centerline of channel to centerline of bridge) and angles of 25 degrees and 29 degrees for alignments X10(B) and X11(C), respectively. The main span crossing of the Detroit River on alignments X10(A) and X10(B) include options that clear span the river and options with one of the main span towers in the river. For alignment X11(C), the required horizontal navigation clearance occupies essentially the width of entire waterway. Therefore for this option the piers are removed from the waterway, away from potential ship impact effects, and all X(11)C options clear span the river. The combination of these configurations result in the range of main span lengths being considered for the Detroit River Crossing in **Table 5**.

After consultation with the US Coast Guard and Transport Canada as well as a practical observation of shipping traffic patterns the placement of a pier on the US side of the river, near the mouth of the Rouge River, was determined to not be practical and was dropped from further consideration. The **Table 5** includes the options with U.S. river piers but these will not be discussed in the remainder of the document.

*Table 5. Summary of Main Span Lengths and Bridge Types*

Alignment	Type Study Option/ Sub-Option		Main Span (m) River Pier (CAN/US)	Bridge Type Cable-Stayed (C) Suspension (S)	Suspension Bridge Side spans Suspended (S) Unsuspended (U) (CAN/US)
X10(A)	1	Option 1a	1,300	S	U
	2	Option 2a	925 (CAN)	S	U/S
		Option 3a	1,000 (US)	S	S/U
	3	Option 4a	925 (CAN)	C	S
X10(B)	4	Option 1a	860	C	S
	5	Option 2a	600 (CAN)	C	S
		Option 3a	650 (US)	C	S
	6	Option 4a	870	S	S
	7	Option 5a	870	S	U
	8	Option 6a	600 (CAN)	S	S
Option 7a		672 (US)	S	S	
X11(C)	9	Option 1a	750	C	S
	10	Option 2a	750	S	U
	11	Option 3a	750	S	S

The resulting span range of 600 m to 1,300 m (1,968 ft to 4,265 ft) has only two viable bridge types. Suspension bridges can be utilized throughout this entire range, while cable-stayed bridges have a practicable span range to about 1000 m (3,280 ft).

### 2.4.1. Cable-Stayed Bridges

The current world record span for the cable-stayed bridge type is 890 m (2,920 ft) for the Tatara Bridge in Japan, soon to be replaced by the Stonecutters Bridge in Hong Kong, with a span of 1,018 m (3,340 ft), and the Sutong Bridge in China, with a span of 1,088 m (3,570 ft). The six longest cable-stayed bridge spans in the world are shown in Table 6 below. In North America, the current longest cable-stayed bridge is the Cooper River Bridge in Charleston, South Carolina with a main span of 471 m (1,546 ft), opened in 2006. The John James Audubon Bridge over the Mississippi River in St. Francisville, LA with a main span of 483 m (1,583 ft) is currently under construction.

*Table 6. Selected Cable-Stayed Bridges in the World*

<i>Year</i>	<i>Name</i>	<i>Location</i>	<i>Span (m)</i>	<i>Span (ft)</i>
Under Const.	Sutong	China	1,088	3,570
Under Const.	Stonecutters	Hong Kong	1,018	3,340
1999	Tatara	Japan	890	2,920
1995	Pont de Normandie	France	856	2,808
Under Const.	Second Incheon	South Korea	800	2,625
2005	Third Nanjing	China	648	2,126
Under Const.	St. Francisville	St. Francisville, LA	482	1,580
2006	Cooper River	Charleston, SC	471	1,546

For this study the cable-stayed bridge type is considered viable for the range of 600 m through 1000 m, meaning that the cable-stayed bridge type is viable for all alignments and options except the clear spanning of the Detroit River on alignment X10(A). This option requires a 1,300 m span, nearly 30% above the world record span for this bridge type, and is not considered a practical alternative.

The specific cable-stayed bridge options that are to be evaluated are shown in Table 5. They range from a 600 m span for Type Study Option 5, which would be a new North American record span length, to 925 m for Type Study Option 3, which would be one of the longest cable-stayed bridges in the world.

### 2.4.2. Suspension Bridges

The current world record span for the suspension bridge type is 1,991 m (6,529 ft) for the Akashi-Kaikyo Bridge in Japan, although the new Messina Straits bridge in Italy, currently under design, is planned to have a 3,300 m (10,827 ft or just over 2 miles) main span. Selected suspension bridge spans, including some of the longest in the world and those referenced herein, are shown in the Table 7 below. In North America, the current longest

suspension bridge is the Verrazano Narrows in New York City with a main span of 1,298 m (4,260 feet), opened in 1964.

*Table 7. Selected Suspension Bridges in the World*

<i>Year</i>	<i>Name</i>	<i>Location</i>	<i>Span (m)</i>	<i>Span (ft)</i>
1998	Akashi-Kaikyo	Japan	1,991	6,529
	Izmit Bay		1,668	
1998	Great Belt	Denmark	1,624	5,328
2005	Runyang	China	1,490	4,888
1981	Humber	United Kingdom	1,410	4,625
1999	Jiangyin	China	1,385	4,543
1997	Tsing Ma	Hong Kong	1,377	4,518
1964	Verrazano Narrows	New York, NY	1,298	4,260
1937	Golden Gate	San Francisco, CA	1,280	4,200
1957	Mackinac	Mackinaw, MI	1,158	3,800
1950/ 2007	Tacoma Narrows 1 & 2	Tacoma, WA	853	2,800
2004	Carquinez	California	728	2,400
1929	Ambassador	Detroit, MI	564	1,850

For this study the suspension bridge type is considered viable for all of the span ranges under consideration. While the longest span range under consideration, 1,300 m at X10(A), is indeed a long bridge, it is well within the range of proven suspension bridge technology and considered viable.

This span range does not represent a significant technological advancement or world record length. Other bridge options in the 750 m to 850 m (2400 ft to 2800 ft) are more common in the U.S, with both the Carquinez Straits and Tacoma Narrows projects being constructed or under way, since 2000.

### **2.4.3. Additional Variations**

Suspension bridge configurations can incorporate suspended and/or unsuspended side spans and cable-stayed bridge configurations can have intermediate piers in the side spans to stiffen the pylon. **Table 5** lists the 11 main bridge options that have been developed for this report based on span lengths and bridge type. Other structural sub-arrangements will be developed and studied in subsequent project phases.

## **2.5. Design Requirements**

For the main structure crossing the Detroit River, the design codes of both U.S. and Canada will apply. The project will be developed using the International System of Units (SI) – (metric). The full design criteria for the Type Study phase is contained in **Appendix B**.

### **3. Navigation**

#### **3.1. Ports of Interest**

##### **3.1.1. U.S.**

In the U.S. the Detroit River and its dockages are within The Port of Detroit which is under the jurisdiction of the US Coast Guard, under the Department of Homeland Security, overseen by Captain of the Port at Detroit, Michigan. Each dock, or terminal, is under strict control of the Captain of the Port for security purposes and the access to those docks is strictly controlled. The following are dockages in the project vicinity [3]:

##### Detroit River

- US Steel Corp, Zug Island Stone Dock, MP 19.6 – approximately 800 feet above mouth of Short Cut Canal.
- US Steel Corp, Zug Island Ore Dock No. 1, MP 19.8 – approximately 1,300 feet above mouth of Short Cut Canal.
- US Steel Corp, Zug Island Docks Nos., MP 20.1 – approximately 2,800 feet above mouth of Short Cut Canal.
- Hazardous Materials Truck Ferry, Detroit Landing, MP 20.5 – immediately above entrance to the Old Channel, Rouge River.
- McCoig Corp., MP 20.6 – 600 feet above entrance to the Old Channel, Rouge River.
- LaFarge Corp., Detroit Terminal, MP 20.7.
- U.S. Army Corps of Engineers, Detroit Area Office Slip and Mooring, MP 21.1.
- City of Detroit, Mistersky Power Station Wharf, MP 21.4.
- Motor City Building Materials, Summit Street Wharf., MP 21.6 (currently not operated).
- Detroit Marine Terminal (Detroit-Wayne County Port Authority), MP 21.9.

##### Rouge River, Old Channel

- Detroit Coke Corp., right bank, MP 0.0
- US Steel Corp, Zug Island Ore Dock No. 2, right bank, MP 0.1

Although not considered a dockage there is a recreational boat slip adjacent to the south edge of Fort Wayne on DTE property.

### 3.1.2 Canada

In Canada the Detroit River and its dockages are within The Port of Windsor.



Figure 5. Port of Windsor Terminals.

1.	Canadian Salt Company (Ojibway)	8.	Southwestern Sales (West Dock)	15.	Lafarge Construction Materials
2.	ADM Agri-Industries	9.	Canadian Salt Company (Sandwich)	16.	Adams Cartage
3.	Canadian Maritime Transport (Detroit-Windsor Truck Ferry)	10.	Vacant port facility	17.	Mill Cove Marina
4.	Morterm Limited	11.	Coco Terminal	18.	Essroc Italcementi
5.	Not in use	12.	K- Scrap	19.	Dunn Paving
6.	Windsor Port Authority Land	13.	Sterling Marine Fuels		
7.	Clark Keith Hydro Dock	14.	Blue Circle CBM		

An existing dock located adjacent to the Brighton Beach Power plant which could be impacted by Crossing X10(A). However, the dock has not been used since the fuel for the generating station was changed from coal to natural gas.

Crossing X10(B) could impact a dock located adjacent to the SW Sales property, particularly if a pier is constructed in the water. SW Sales relies on the docks for importing stockpiles of aggregate. Additional discussion with the property owners is required to confirm impacts to their shipping operations. Shipping operations of a dock located adjacent to Coco Paving will not be impacted by the bridge since it is located away from the X10(B) and X11(C) alignments. Another docking facility is located adjacent to Sterling Fuels. Although no piers are proposed in the water at this location, additional discussions with the property are required to confirm that Sterling Fuels and Crossing X11(C) can co-exist.

### 3.2. Marine Traffic

The Detroit River extends approximately 52 kilometers (32 miles) from its mouth at Lake Erie to Lake St. Clair. One of the busiest inland waterways in the world, the river carries more shipping traffic than any other river in North America. The principal ports on the Detroit River are at Trenton, Wyandotte, and Detroit, Michigan, and Windsor, Ontario. Deep draft facilities have been developed throughout the length of the river. For the year 2003, freight traffic reported for the limits of the Detroit River totaled 58,024,443 tonnes (63,961,000 tons).

Marine traffic on the Detroit River in the vicinity of the proposed bridge sites is based on information provided by the Canadian Coast Guard (CCG) Marine Communications and Traffic Services (MCTS) program which manages the movement of vessel traffic in Canadian waterways. The Detroit River and St. Clair River fall within the Sarnia jurisdiction of the MCTS.

For the years 2003 to 2005, the number of vessel transits average 6,000 up-bound (northbound) and 5,600 down-bound (southbound) per year. For this period, approximately 57% of vessels are characterized as large bulk carrier or tanker ships. The remaining 43% are smaller vessels such as tugs, passenger vessels, Coast Guard vessels and fishing vessels. It should be noted that the actual number of vessels transiting past the proposed bridge locations may be less than these reported numbers since not all vessels travel the full length of the river.

Vessel characteristics for the large bulk carrier and tanker ships using the waterway were established based on discussions with MCTS staff and confirmed by local shipping companies. Generally, the larger size bulk carrier and tanker ships are approximately 300 m (984 ft) in length and have a beam of 30 m (98 ft). These vessels have loaded drafts of approximately 9 m (30 ft) consistent with the maximum draft of the waterway which can vary between 8 and 9 m (27 and 30 ft) depending on water levels. The cargo capacity of these vessels, or Deadweight Tonnage (DWT), is approximately 65,000 tonnes (72,000 tons).

### **3.3. Navigation Channel and Clearances**

After consultation the U.S. and Canadian Coast Guard advised that the navigation clearances of the current Ambassador Bridge be maintained. The following navigational clearance data was compiled from the Ambassador Bridge as-built record plans:

- Vertical Clearance of 46.33 m (152 ft) from High Water Line, (47.43 m (155.6 ft) to MWL), over a 30.48 m (100 ft) width near middle of the channel (243.23 m (798 ft) from N Tower and 260.60 m (855 ft) from S. Tower)
- Vertical Clearance of 40.54 m (133.0 ft) to HWL at North (U.S.) Harbor Line
- Vertical Clearance of 40.69 m (133.5 ft) to HWL at South (Canadian) Harbor Line
- Vertical clearances to be maintained over full width of River
- Horizontal Clearance of 534.31 m (1,753 ft) from Harbor Line to Harbor Line

The Ambassador Bridge plans do not state which vertical survey datum was used. In consultation with staff at National Oceanic & Atmospheric Administration (NOAA), they indicated that the best approximation is the Second General Adjustment of 1903. This datum was converted to the DRIC project vertical datum which is NAVD88. According to the U.S. National Geodetic Survey NAVD88 is equivalent to IGLD85 in this location. Therefore the High Water Level elevation, as shown on the Ambassador Bridge plans, is 175.439m (IGLD85). Finally, the Ambassador Bridge is at a skew angle of 10 degrees to the channel centerline. **Figure 3** shows the proposed project navigation envelope, which is perpendicular to the channel centerline.

As the bridge options were developed it was apparent that at corridor X10 this proposed navigation envelope was much narrower than the channel width. Placing both the main piers in the river, while maintaining the proposed navigation envelope, would significantly reduce the main span length and, would reduce the overall structure length, resulting in potential cost savings.

Initial consultation with agencies did not identify a prohibition against piers in the water; however, placing two piers in the water would increase costs of marine construction and pier protection. Therefore, it was proposed to shift the channel center line, placing one pier on land and one in the river. This arrangement could also allow sufficient distance behind the river pier to allow some navigation between the main pier and shore, in order to access shoreline industries, if necessary.

### **3.4. Pier Protection**

Pier protection was evaluated in accordance with the requirements of the AASHTO design specifications. The Canadian Highway Bridge Design Code has similar provisions that were adapted from AASHTO. The vessel collision specification requires that bridge elements within the navigable portion of the waterway shall be designed for vessel collision based on site specific waterway and vessel characteristics. Four of the Main Bridge Options under consideration will require placing a single pier in the river adjacent to the navigation channel. These include Type Study Options 2 and 3 at Crossing X10(A); and Type Study Options 5 and 8 at Crossing X10(B).

The design for pier protection is based on the large bulk carrier or tanker ship as described under **Section 3.2** and an assumed vessel transit speed of 8 knots. Using the AASHTO specifications for vessel collision, the risk acceptance criteria would require that the piers be designed for an equivalent static vessel impact force of 28,000 kips. Although technically feasible, designing pier foundations for a force of this magnitude is not considered practical.

Alternatively, a physical protection system consisting of an arrangement of large diameter dolphins is viable for pier protection. The designs of individual dolphins are based on energy absorption principles and require deformation and displacement of the dolphin during vessel impact. A typical dolphin required for the design vessel is shown in the figure below. The proposed arrangement of dolphins is shown on the General Plan and Elevation sheets. This arrangement consists of four to six dolphins, each 20 m (66 ft) in diameter, spaced along the edge of the proposed navigation channel adjacent to the main pier.

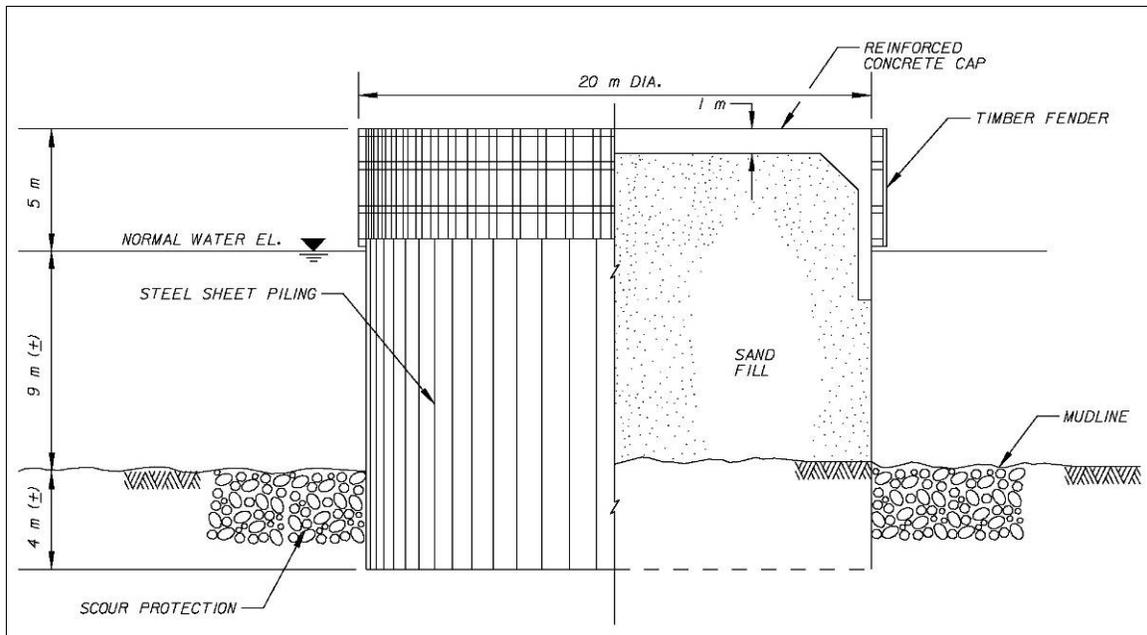


Figure 6. Pier Protection Dolphin.

## 4. Aviation

Due to the height of bridge towers and pylons being considered impacts to local airports must be evaluated. Three airports are reasonably close to the bridge corridors; Detroit City Airport, Windsor Airport, and Grosse Ile general aviation airport. Windsor Airport is located in the Canadian Province of Ontario, approximately 8 km (5 miles) east of the Detroit River and 5 km (3 miles) south-east of downtown Windsor.

An assessment was performed based upon the airspace configuration from the surrounding airports in which the proposed bridge structures would be located. On that basis, a more detailed analysis was performed which concludes that the bridge configurations currently being assessed under the DRIC study would not encroach upon the Federal Aviation Administration's (FAA) airspace design criteria.

Additionally, the Transport Canada Aeronautical Information Services Program Officer has been contacted regarding a confirmation of the clearances and an assessment of the potential marking and lighting requirements for the tower structures. At the time of writing of this report, confirmation of clearances has not been received. This matter will be followed-up as an early priority during Concept Engineering phase.

With an overall elevation of 445m (1,460 ft) above Mean Sea Level (MSL), Cable-Stayed Bridge pylons at X10B provide the least amount of vertical clearance (1m) to the overlying airspace associated with the current departure procedures from Windsor Airport. Therefore, increasing the overall height of the eastern pylon by more than one meter (three feet) would create a hazard to air navigation. Furthermore, relocating the proposed pylon structure further to the north or east could result in an encroachment of the overlying airspace. Based on the

current configuration of the proposed bridge structures, it is determined that there would not be an impact to existing regional airspace procedures. A detailed assessment may be found in the Technical Memorandum by the Corradino Group, dated October 18, 2006 [4].

It is noted that the ongoing bridge development will need to continue to coordinate with the airport, both for permanent clearances and temporary conditions during construction.

## **5. Geology and Seismicity**

### **5.1. Geological Conditions**

#### **5.1.1. Summary of Bedrock Information (U.S.)**

The proposed crossing corridor is located at the geologically termed southeast margin of the Michigan Basin and within the Erie-Huron lowland. The Michigan Basin is termed as such due to the structural basin shape of the bedrock, in which layers of Paleozoic era sedimentary rock that overlay the Precambrian Basement Complex, dip inwards to the center of the Lower Peninsula of Michigan from each direction as a series of bowls.

The Michigan Basin is bounded on the west by the Wisconsin Arch and Wisconsin Dome; on the north and northeast by the Canadian Shield; on the east and southeast by the Algonquin Arch in Ontario and the Findlay Arch in Ohio; and by the Kankakee Arch in northern Indiana and Illinois. The Michigan Basin has undergone several periods of subsidence and rebound during the Paleozoic Era, creating a complex interbedding of various sedimentary rocks.

Based on the position of Detroit, Michigan, and Windsor, Ontario along the southeast rim of the Michigan Basin, the Paleozoic rocks that comprise the basin in this area typically dip to the northwest, with each formation being buried by successive younger formations in the direction of the dip. The regional dip is slight, and is estimated at approximately 6 to 10 m per kilometer (30 to 50 feet per mile).

The topography of the bedrock surface within the area is somewhat variable and characterized by numerous irregular features in the bedrock surface. The features are believed to have developed before the Pleistocene Epoch and subsequently were modified by repetitive glacial action. The bedrock features include the existence of ancient stream valleys that cut the bedrock surface. Based on historical information, the bedrock features are understood to be fairly broad, and become narrow as they reach the terminus of the Erie / Huron Lowlands.

These strata are seamed and fissured with vertical joints that permit movement of ground water. Where carbon dioxide dissolved within these groundwater-filled cracks, solution voids typically developed within the interbedded limestone and dolomitic limestone beds. Both the limestone and dolomitic formations are known to contain dissolved sulfides, which can produce hydrogen sulfide gas upon exposure to atmospheric conditions.

Hydrogen sulfide gas in the proposed crossing area has a history of causing nuisances and toxic conditions during tunneling operations and deep excavations, causing injury and sometimes death to construction workers. The natural decay of organic compounds that also existed within the ancient seas became trapped within cavities formed in the limestone and dolomites and is evident today as petroleum. Small amounts of petroleum found within the limestone and dolomite tends to cause discoloring, staining, and produce associative odors. Modern construction techniques can mitigate these concerns if appropriately identified.

### **5.1.2. Summary of Overburden Information (U.S.)**

The bedrock along the project corridor is overlain by glacially deposited soils (drift), which have been deposited either directly by glacial ice (till), by glacial meltwater streams (glaciofluvial), or by glacial lakes (lacustrine deposits). The upper soil formations along the alignment generally consist of a relatively thick mantle of Wisconsin aged lacustrine clays (10,000 to 50,000 years ago) that, with the exception of the near-surface deposits, are typically medium to stiff in consistency. The upper 3 to 6 m (10 to 20 feet) of these deposits have been desiccated during historical low water periods, resulting in soils of very stiff to hard consistency near the surface. The clay soils frequently contain intermittent sand and gravel layers.

The lacustrine deposits are typically underlain by a thin layer of highly over-consolidated glacial till, generally consisting of sand, silt, and gravel within a matrix of clay and usually overlies the bedrock formation. Depending on the amount of clay binder contained in this deposit, the material may range in nature from cohesive to granular and can also contain calcium carbonate producing a cemented condition. Given the glacial origins of the layer, occasional cobbles and large boulders are typically present in this layer.

The total drift along the X10 and X11 corridors varies in thickness from approximately 27 to 30 m (90 to 100 feet).

#### **5.1.2.1 Crossing X10 and X11 (U.S.)**

In the proposed corridors, approximately 100 m (325 feet) of interbedded limestone and dolomites (Dundee Limestone Formation and Detroit River Group) comprises the bedrock immediately below the overburden at approximately Elevation 148 m (EL 486 feet). Based on the historical data, the Dundee limestone, anticipated directly below the overburden, in this area is higher permeability, typically in the range of  $10^2$  to  $10^4$  cm/sec, with the highest permeabilities near the soil/rock interface.

The highly over-consolidated glacial till covers the bedrock by a thickness on the order of 2.5 to 3 m (8 to 10 feet) is expected. Soft ground soils generally consist of soft to stiff silty clay away from the riverbank. At the river's edge, granular soils are expected with varying amounts of silt, clay, and gravel. Overlying the native granular soils, fill soils of varying type and consistency are expected, with the potential for environmental contamination and deleterious material. The bottom of the Detroit River within the navigation channel is expected to be approximately

Elevation 164.5 m (540 ft), resulting in soft ground cover on the order of 16.5 m (54 ft).

### **5.1.3. Summary of Bedrock Information (Canada)**

Within the Windsor area, the bedrock geology consists of an evaporate-carbonate sequence of rock formations. These include the Silurian Salina formation, the Devonian Bass Islands dolomite, the Detroit River Group, the Dundee Formation and the Hamilton Group, respectively, with decreasing age and closer proximity to the ground or bedrock surface. The surface of the bedrock, beneath the overlying sediments, is relatively flat except a significant depression in the vicinity of the Windsor Airport. The depression may represent a dissolution collapse of either the underlying carbonates or the lower Salina salt beds.

Devonian Age bedrock of dolomite, shaly limestone, limestone and sandstone extend from the bedrock surface, found at depths of between 20 and 40 m (66 and 131 ft), to depths of about 160 m (525 ft) below ground level. These bedrock formations are underlain by the Salina Group of formations that includes thick salt beds at depths of about 270, 300 and 400 m (886, 984, and 1,312 ft) below the ground surface. It is also known that relatively small volumes of petroleum are found within the limestone and dolomite stratum.

Groundwater within the bedrock is in some areas known to be under artesian pressures. In these areas artesian pressures may be on the order of 2 to 3 m (6 to 10 ft) above the river level. Groundwater from within the bedrock is likely to be naturally corrosive.

It is also known that in some areas the groundwater contains sulfide that will be liberated from solution and become hydrogen sulfide gas at normal atmospheric pressures. Hydrogen sulfide gas is toxic at low concentrations. Methane gas has also been encountered during excavations into both soft ground and bedrock in the Detroit Windsor area. Methane gas can present an explosion hazard if not adequately controlled during construction.

### **5.1.4. Summary of Overburden Information (Canada)**

The study area is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plains. Within this region, Essex County and southwestern the part of Kent County are normally discussed as a sub region known as the Essex Clay Plain. The clay plain was deposited during the retreat of the ice sheets (late Pleistocene Era) when a series of glacial lakes inundated the area. In general, the ice sheets deposited till in the area of Windsor and Detroit. Depending on the location of the glacial ice sheets and of water in the ice-contact glacial lakes, the till may have been directly deposited at the contact between the ice sheet and the bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of the ice may have been deposited through the lake water (lacustrine). The mineral soil particles typically have a distribution of grain sizes ranging from cobbles to clay.

The major silty clay to clayey silt stratum, typically ranging in thickness from about 20 to 30 m (66 to 98 ft), exhibits a till like structure exemplified by a random distribution of coarser

particles within the primary fine grained silt and clay deposit (this type of deposit is also called "diamict"). In the crossing areas, below frost depth, the near surface clay is generally stiff to very stiff, brown in color and exhibits undrained shear strengths in the range of 50 to 100 kPa or more. This layer is often 2 to 6 m (6 to 20 ft) thick. Underlying this stiff to hard "crust" and below groundwater levels the clayey silt becomes gray in color, is soft to firm and exhibits undrained shear strengths in the range of 20 to 40 kPa.

Surficial layers or pockets of more typical layered lacustrine (lake deposited) silty clay or sand may be encountered overlying the extensive stratum of "till-like" silty clay. Silt and sand deposits on the order of 2 to 4 m (6 to 13 ft) thick are often found near the ground surface in the areas near the western side of Windsor. A relatively thin stratum, on the order of 1 to 6 m (3 to 20 ft) thick, of very dense or hard basal till or dense silty sand or silt is found directly over the bedrock surface.

#### **5.1.4.1. Crossing X10 and X11 (Canada)**

Based on experience in the area the surface soil conditions should be similar for the X10 and X11 crossing sites. The greatest potential for differences between the overburden conditions is likely the thickness of the fill materials that have been placed on the sites. At the time this report was prepared there was not enough data to conclude that the sites were significantly different in this aspect. As the River Bridge foundations will be founded on bedrock and the surface soil will have essentially no influence on the selection of the preferred alignment, it was not considered necessary to obtain additional information at this time.

## **5.2. Brine Wells**

### **5.2.1. U.S. Side**

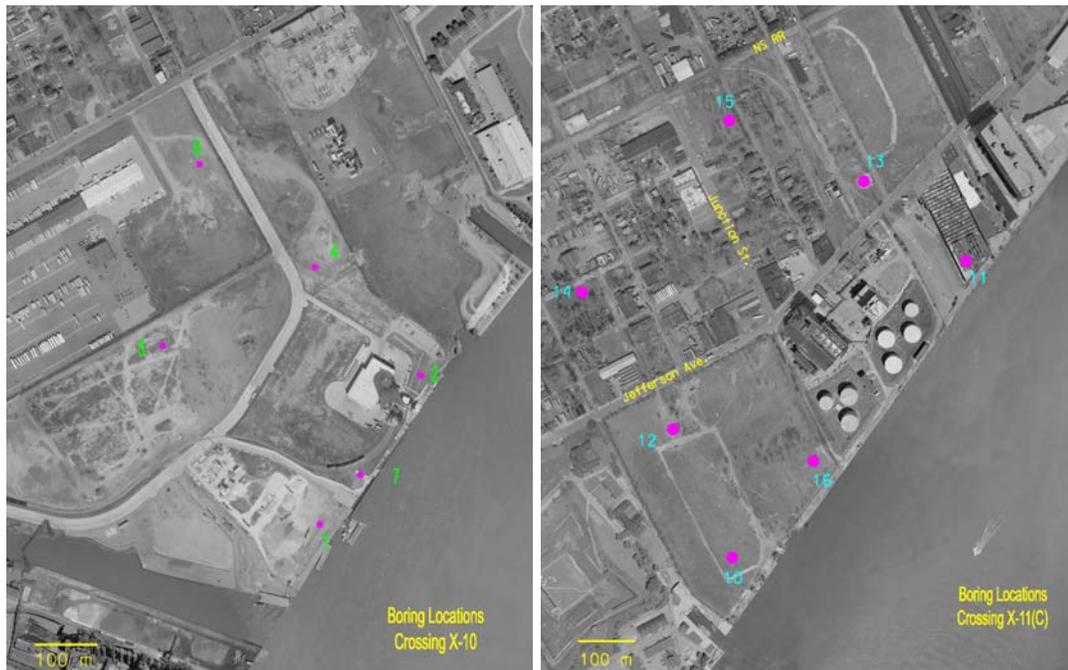
The Michigan Basin is one of the largest areas of halite (salt-NaCl) deposition in the world. Halite has historically been mined either directly in solid form as rock salt or as natural or artificial brine pumped through solution mining wells. The area beneath Detroit and Windsor within the Michigan Basin is currently mined primarily using conventional room and pillar excavation methods. Historically, beginning in the late 1880's, solution mining was used to mine for salt. Solution mining in the proposed crossing areas was generally discontinued in the 1960's as a result of increasing concerns of surface subsidence. Windsor Salt still does some solution mining in the area under modern control methods. Known areas of solution mining were preliminarily identified and discussed in the Geotechnical Evaluation Report [1].

Generally, known solution mining areas are located on Zug Island and to the southern end of the project study area, but the occurrence of unknown brine wells throughout the corridor cannot be precluded as many unknown wells are thought to exist. Further, solution mining companies are known to have owned parcels of land along the river in addition to those where brine wells were documented. Generally, the brine wells extended to depths of 335 m (1,100 ft) to 460 m (1,500 ft) in the area of continued analysis.

In general, solution mining consists of introducing water from the surface down a well casing between an outer casing and a central tube. The brine produced from the salt dissolving in the water is recovered through the central tube. With continued production using this method, solution cavities often coalesce with adjacent cavities to form composite cavities called galleries. When this occurred historically, one or more of the wells were then converted to water inlet wells and the brine was pumped out through other wells in the interconnected system, creating a gallery.

As production continued in the gallery, large spans of unsupported roofs were sometimes created, which in turn could result in sagging, downward flexure, and local separation of rock units resulting in local roof collapse and eventual surface subsidence in some instances. Uncontrolled solution mining near the top of a salt layer commonly left overlying weak or weakened rocks exposed at the top of the cavity, which increased potential for roof collapses. The subsidence and/or collapse would progress upwards as a chimney effect on an angle of up to 10 degrees from vertical from the outside edges of the cavity.

The solution mining areas are of concern for the proposed crossing locations, as they present the potential for future ground subsidence and related adverse effects on elements of the proposed crossing structure. Due to the concerns regarding solution mining an extensive investigation program was developed and is underway concurrently with the bridge type study.



U.S. Deep Bore Hole Locations

The goal of the brine well investigation program is to ensure that the main structure is built on sound bedrock outside the influence of deep brine cavities of a structurally significant size. Specifically, there shall be no catastrophic event or structural damage to the main structure. Therefore, the comprehensive brine well investigation program on the U.S. side

of the project is focusing on ensuring that no brine wells of a size and configuration to cause catastrophic failure or structural damage to the main structure exist within the bridge corridors.

This program involves both forward and inverse geophysical modeling techniques to determine the void sizes that could propagate to the surface. In the U.S. fourteen deep bore holes are being placed, seven at each crossing location, to a depth of 460 m (1,500 ft) to 535 m (1,750 ft). Cross-well reflection imaging and borehole gravity techniques will then be employed to determine if voids are present, and if so, what size, location, configuration, and potential for propagation upward from their present location.

### **5.2.2. Canadian Side**

Salt extraction activities on the Canadian side have been undertaken with two different methods; solution mining and dry mining. Salt extraction by solution mining involves pumping water into wells drilled into the salt formations, dissolving the salt with pumped water, then extracting the salt from the saline water (brine) which is returned to the surface. Dry mining (also called “room and pillar mining”), involved digging mine shafts from the ground surface to the salt bed level, excavation of the salt by drilling or blasting, and transporting the salt to the surface in large buckets or “skips”. Dry mining has only taken place south of X10(A) at the Ojibway mine and is not a concern for the X10 or X11 crossings. Either of these mining methods creates deep cavities that can affect the ability of the overlying rock and soils to carry a foundation load.

There is and has been extensive salt production in the specific vicinity of the proposed crossing of the Detroit River, particularly near the X10(B) alignment using brine wells from about 1901 to 1954. The newer mining (after about 1970) on properties east of both crossing locations is well documented with regard to size and depth of caverns, and with good records related to interconnectivity of caverns. However prior to 1970 there is limited information available and interconnectivity of mines was not well controlled or documented.

In general, removal of salt creates greater stress on the remaining salt. Large roof spans created in caverns can cause sagging of the overlying stratum and downward of the rock around the caverns and result in subsidence of the ground surface. Subsidence rates can vary substantially but rates on the order of 3 mm (0.12 in) per year have been recorded by recent surveys of abandoned caverns and 10 mm (0.4 in) per year over operating or recently operating caverns. The subsidence may eventually cease as a result of the gradual collapse of the cavern and it’s filling up with rock and debris. For relatively large caverns the collapse can be in the form of a sinkhole over a short period of time. One of the most dramatic of these events occurred in 1954 at the Canadian Industries Limited facility in Windsor (located between the X10 and X11 crossings). In this instance, an approximate 300 m (984 ft) diameter bowl-shaped depression developed over the course of a number of years with central settlements on the order of about 50 mm (2 in). Then within the period of a few hours the ground collapsed into a sinkhole about 9 m (30 ft) deep at the center and 150 m (492 ft) diameter. Several buildings were irreparably damaged during the incident. The sinkhole was later filled and the property has been used for open storage and a rail yard.

Similar to the U.S. side, on the Canadian side a brine well investigation is underway to ensure that the main structure and approach structures are built on sound bedrock outside the influence of deep brine cavities of a structurally significant size. A comprehensive brine well investigation program on the Canadian side of the project is focusing on ensuring that no brine wells of a size and configuration to cause catastrophic failure or structural damage to the main structure exist within the bridge corridors.

This program involves drilling deep borings (to 500m depth each) and cross-hole geophysics investigation techniques to identify if cavern void sizes exist that could propagate to the surface. In Canada deep bore holes are being placed, six at each crossing location with the X10(B) locations shown on the accompanying plan and one additional hole to be located to investigate any abnormalities that may be observed from drilling the first six holes. Cross-well reflection imaging techniques will be used between the boreholes to detect and voids or cavities.



Figure 7. Canadian Deep Bore Hole Locations – X10(B).

**5.3. Seismicity**

According to historical Seismic risk maps published by the U.S. Geodetic Survey, the project is located within Seismic Risk Zone No. 1. The historic return period for seismic events is 475

years. The bridge is classified as a “critical” structure. Based on known soil information in the project area the soil profile is Type IV, which will be re-confirmed once site specific deep test borings are performed at the locations of the proposed primary foundation elements.

#### **5.4. Scour**

Scour of the riverbed adjacent to pier foundations and dolphins is not anticipated to be significant. River current velocities are governed by the hydraulic gradient between Lake St. Clair to the north (stage El. 174.4 meters) and Lake Erie to the south (stage El. 173.5 meters). The Detroit River is about 51 km (32 miles) in length. Seasonal fluctuations and weather conditions can affect water elevations and consequently river current velocities. Average river current velocities in the vicinity of the proposed bridge site have been reported as 1.2 knots (low flow conditions), 1.3 knots (medium flow conditions) and 1.4 knots (high flow conditions). Mitigation for long-term scour effects, if required, can be accomplished by armoring the riverbed adjacent to pier foundations and dolphins using rubble rip-rap or designing foundations for scoured conditions. This would be based on riverbed in-situ materials and hydraulic analysis with the structures in place. Short-term scour effects associated with flow conditions under flood events is not anticipated.

### **6. Foundation**

The very heavy foundation loads for the main river crossing piers require deep foundations to carry these loads into bedrock. This section discusses potential deep foundation alternatives.

#### **6.1. Drilled Shafts**

Drilled large diameter concrete filled shafts are the most common foundation type to bear the heavy foundation loads in competent bedrock. The drilled shafts should extend through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded into the underlying limestone bedrock formation, resulting in depths of approximately 35 m (115 feet). This will minimize uncertainties in the design by providing a uniform and reliable bottom pier elevation bearing on competent rock.

#### **6.2. Driven Piles**

The deep foundation system may be planned to consist of concrete-filled steel pipe piles or H-piles bearing in competent bedrock. The piles should be driven through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded into the underlying limestone bedrock formation, resulting in depths of approximately 35 m (115 feet). This will minimize uncertainties in the design by providing a uniform and reliable bottom pier elevation bearing on competent rock.

Concrete-filled pipe piles are considered less likely than other foundation systems (such as H-Piles) to allow leakage of artesian groundwater around the pile / soil contact. Use of concrete-filled steel pipe piles will also allow greater freedom in sequencing pile driving operations, because they can be driven from a higher ground surface elevation, then cut-off at the design elevation using an internal cut-off tool, with the remaining waste section removed. However, the use of driven piles may not be acceptable due to the potential for damage while driving into and through the glacial till soils and due to the large number of driven piles required to transfer

the very heavy main pier foundation loads into the bedrock. This foundation type should be further investigated for the approach structures.

### **6.3. Sunken Caissons**

Large caissons have proven to be economical in certain conditions for marine piers of long span structures. A caisson consists of a buoyant steel "cutting edge" that is fabricated off-site and towed into position. The cross section is such that dredge wells are created in a grid or other configuration that will later allow access from the top all the way to the riverbed. Atop the cutting edge, reinforced concrete walls are constructed, the weight of which sinks the cutting edge down through the water. As the cutting edge sinks into the riverbed, barge-mounted clamshell buckets excavate soil from the riverbed via the dredge wells, allowing the cutting edge to push down into the soil. This process proceeds until the cutting edge reaches a predetermined depth.

Some examples of caissons are cited below for comparison purposes:

- Two 18,000 tonne (20,000 ton) caissons were used for the foundation of the New Oresund Bridge between Copenhagen and Malmo in Sweden/Denmark. Each of these caissons is 35 m by 37 m (115 ft by 121 ft) in plan and 22.5 m (74 ft) high. The Oresund Bridge is a cable-stayed bridge with a main span of 490 m (1608 ft) and tower height of 203.5 m (668 ft). These are comparatively small caissons.
- The new Tacoma Narrows Bridge in Tacoma, Washington used a rectangular caisson 39.6 m long, 24.4 m wide (130 ft by 80 ft) and about 60 m (197 ft) tall, founded about 18 m (60 ft) below the seabed.
- The world's longest suspension bridge, Akashi-Kaikyo, used circular caissons. The construction started in the dry-docks with the assembly of a doughnut-shaped cylinder of steel each measuring 80 m (262 ft) in diameter, 70 m (230 ft) in height and weighing 15,000 tonnes.
- The Ambassador Bridge main tower foundation was constructed of two cylindrical sunken caissons consisting of approximately 6,100 thousand cubic meters (8,000 thousand cyds) of concrete each. The anchorages were constructed by using sunken caissons as well consisting of about 14 to 17 thousand cubic meters (18-22 thousand cyds) of substructure concrete each.

A similar approach of large diameter sunken caissons is also feasible for land based piers. Such caissons were utilized for the existing Ambassador Bridge anchorages. Although there are numerous factors related to geological and seismic conditions that will have a major effect on the size of the caissons, rough proportioning shows that the caissons for this bridge would likely be slightly larger than the Tacoma Narrows caissons.

The caissons require specialized construction through the anticipated upper granular zones. The potential for contaminated and deleterious material, the artesian pressure levels and contaminated groundwater could present significant risks.

## **6.4. Braced Excavations**

Anticipated excavations for the bridge foundations and appurtenant elements will extend through fill soils and into the underlying granular and soft cohesive soils. Open excavations through the fill soils may be possible within the upper 1.5 to 3.0 m (5 to 10 ft) using side slopes if adequate space exists. Due to groundwater within the granular soils, open excavations through the granular soils will be difficult without groundwater control and/or use of earth retention systems. Based on experience in the project area excavations within the soft cohesive soils are very difficult due to squeezing soil conditions and basal instability.

Substantial temporary earth retention systems will be required to excavate within these soils, increasing in capacity and cost with depth and plan dimensions. Excavations near or into the bedrock will require substantial rock grouting to control groundwater and associated hydrogen sulfide gas infiltration from the bedrock. Even with grouting, some infiltrations should still be expected. Using dewatering of the rock to control infiltrations has been attempted with limited success.

For the suspension bridge anchorages, the use of longitudinal shear walls remains a viable option. These would be cast-in-place reinforced concrete walls, cast inside braced excavations and would serve to transmit the tension of the main cables to bedrock. It is of note that the existing Ambassador Bridge anchorages are founded on similar structures, the difference being that in lieu of the braced excavation method, open caissons were used as a construction method, the walls of which serve as shear walls.

## **7. Main Bridge Cross Sections**

### **7.1. General Configuration**

Traffic across the international border presents unique factors that must be considered in conjunction with traditional design standards in an effort to establish the appropriate cross section. Long-term performance of the proposed Detroit River International Crossing will be affected by several critical elements:

- 1) Traffic Capacity
- 2) Traffic Safety
- 3) Operational Capacity
- 4) Flexibility/Expandability

These factors are discussed in greater detail in the Draft River Bridge Cross Section Technical Memo [2]. In summary, the consideration of an appropriate bridge cross section is heavily influenced by the desired level of performance balanced with economic considerations. Performance and economy are evaluated within the context of a reasonable design horizon. The international border crossing presents unique performance needs in order to maintain the flow of goods and travelers. A harmonization of standards between the U.S. and Canadian portions of the project is also required.

Many cross sections detailed in the Technical Memo were considered. At the time of printing a final decision on the cross section had not been made and investigations and discussions are ongoing by the Partnership in this regard. For the purposes of this study the following cross

section was used: six (6) 3.75m (12'-4") travel lanes, a 3.0m (9'-10") right shoulder, TL-4 exterior railing, and a single 1.6m (5'-3") sidewalk, for pedestrian use only interior to the suspension system, bicycle traffic will be allowed to use each right shoulder – which will be striped for one way bicycle traffic, as shown in **Figure 2**. A closed box system with vertical hangers is shown, however, other structural deck configurations and cable arrangements are discussed in other sections of this report.

## **8. Suspension Bridge Options**

### **8.1. Description of Suspension Bridge Options**

#### **8.1.1. Layout**

As discussed in **Section 2.4** several options may be considered for the suspension bridge layouts. One fundamental consideration is whether or not one or both of the side spans are to be suspended. For suspension bridges where spanning a significant length is necessary, for instance where the main pier is in the water, the side spans are often suspended. Where the side span is on land and there are no obvious physical constraints, it is usually more economical to construct an un-suspended side span. This side span can be much shorter than a suspended side span, and as a result allows the horizontal alignment of the traveled roadway to begin curving at an earlier point. The unsuspended side span is constructed on piers with spans on the order of 100 m (328 ft) in length.

The following subsections have been prepared for the suspension bridge options discussed in **Section 2.4**. It should be noted, however, that variations remain available to avoid possible obstructions on land. For instance, side span lengths may be varied significantly, or cable bents or tie downs could be used to alter the cable angle at the end of the side spans to allow flexibility in the location of the anchorages. As the project moves forward, such refinements may be considered as a matter of economy or appropriateness for the site.

#### **8.1.2. Towers**

The two vertical legs of the two suspension bridge towers would be hollow reinforced concrete. This construction method has proven much more economical than steel construction. The tower would be an H-shape with horizontal bracing struts. Lateral reactions of the superstructure against the tower legs from wind loads would necessitate a heavier cross section below the roadway. Tower legs would likely be inclined to allow the main cable saddles to be positioned vertically above the suspender connection at deck level, and along the centerline of the tower leg to minimize flexural forces. Architectural treatments may include chamfering, sunken panels or similar treatments.

The horizontal struts, of which there may be two or three, would likely be of post-tensioned concrete to realize material savings. The struts would also be hollow box members in section, allowing maintenance personnel to move between tower legs. The outside faces of the struts may include architectural treatments to reflect the chosen theme from the CSS workshops.

Towers are typically equipped with maintenance elevators inside one tower leg and a combination of stairs and ladders in the other. Grounding is also provided to protect the structure from lightning strikes. A surface treatment may be applied to the outside of the tower legs to ensure a uniform appearance and to provide an additional layer of corrosion protection.

### **8.1.3. Cable Anchorages**

The cable anchorages resist the tension of the main cable through the mass of the concrete in the anchorage, and given the soil properties at the site, a suitable deep foundation. A system of strand shoes, anchor rods and anchor frames buried in the anchor block of the anchorage transfer the tension of the cable to the concrete. The anchor block should be maintained above the water table to protect the cable anchoring system from corrosion. The anchorages will be mainly above grade, providing an opportunity for architectural treatments.

### **8.1.4. Cable System**

The major components of the cable system are the main cables, cable bands, saddles, and suspender ropes.

#### **8.1.4.1. Main Cable**

There are two methods available to make the main cables for this scale of bridge. In the first method, the main cable is constructed on-site by “air spinning” individual wires (usually 5 mm dia.). The wires are laid parallel to each other and span over the towers and anchor at the two cable anchorages. This method has been used since the Brooklyn Bridge. More recently, this method has been used on the Carquinez Bridge (wire diameter: 4.978 mm, No. of wires per strand: 232, No of strands: 37), the Tacoma Narrows Bridge (wire diameter: 4.978 mm, No. of wires per strand: 464, No of strands: 19), and the Storebaelt Bridge.

The second method of main cable construction consists of stringing prefabricated parallel wire strands. This method was used in the Kanmon Bridge, Japan (wire diameter: 5.04 mm, No. of wires per strand: 91), the Ohnaruto Bridge, Japan (wire diameter: 5.37 mm, No. of wires per strand: 127) and the Akashi Bridge, Japan (wire diameter: 5.23 mm, No. of wires per strand: 127).

For this bridge, both schemes are feasible and present unique challenges, opportunities and risks. Implications for the anchorage configuration, construction risks, construction schedules, advantages/ disadvantages, etc. must be carefully considered during future phases of the project before selecting either method. A further investigation at the stages of conceptual and detail design may be based on these factors, economy, and quality control.

It is becoming more common for suspension bridges of this magnitude to utilize a dehumidification system for the main cable and anchorages. This provides an added mitigation measure against corrosion of the main cable and anchorages. Visual inspection of the main cable is typically accomplished by removing wrapping wire at

specific locations and splaying the strands out by driving wooden wedges into the cable.

#### **8.1.4.2. Cable Bands**

Cable bands clamp to the main cable and function to receive the suspender ropes. These are typically cast steel, but may also be fabricated weldments, though current market trends would likely make fabricated weldments an unattractive option. Cable band components are not overly large and may be competitively manufactured both in North America and overseas.

#### **8.1.4.3. Saddles**

Cable saddles are used at the tower tops and where the cable splays from its circular cross section into individual strands at the anchorages. The saddles serve to cradle the cable at support points.

Two distinct fabrication schemes are employed to produce saddles, the first method being the more traditional approach. The first method is to cast all elements of the saddle in one piece, although for construction handling allowances or to facilitate casting, the saddle may be fabricated in two or three sections and bolted together. The advantages of this approach include the monolithic and homogeneous characteristics of the resulting saddle.

As saddles become larger, it becomes increasingly difficult to achieve a high quality uniform casting. In such cases a second method may be used in which only the trough is cast steel while steel plates are used for base, stiffener, etc. All components are joined by complete penetration welds. The first scheme was used in the Carquinez and Tacoma Narrows Bridges. The Storebaelt and Akashi bridges used the second scheme. Further investigation at the stages of conceptual and detail design will be based on economy, construction schedule and ease of procurement.

#### **8.1.4.4. Suspender Rope**

Suspender ropes are the links between main cable and suspended deck. These are replaceable items with a service life on the order of 50 years. Provisions for their replacement include details that allow for connecting temporary jacks for destressing the suspender ropes during replacement operations, as well as confirming the superstructure will perform adequately with a suspender rope temporarily removed. The wire rope can be manufactured competitively both in North America and overseas.

### **8.1.5. Deck System**

For suspension bridges, two different deck systems are generally considered, steel orthotropic box girders and stiffening trusses.

#### **8.1.5.1. Box Girder**

Orthotropic steel box girders have been used in long-span suspension bridges since the Severn River Suspension Bridge was built in Wales, England in 1966 and more recently in the U.S. for the Third Carquinez Straits Bridge. An orthotropic box girder is

comprised of a thin steel shell or "skin", stiffened internally with longitudinal ribs and transversely with bulkheads spaced at regular intervals that also support the ribs.

Advantages of the system include a shallow structural depth which translates to shorter approach pier heights and reduced approach grades, efficient use of materials, some advantages for maintenance (particularly maintenance painting), utilities enclosed within the structure and hence protected from the elements, favorable aerodynamic behavior and clean aesthetic lines.

Disadvantages of the system include fabrication complexity, lessened material efficiencies with respect to European box girders due to minimum plate thickness requirements in the U.S. design codes, and increased amounts of field welding.

The orthotropic box girder is well-suited for use with suspension bridges. Suspension bridge stiffening girders are typically designed to be moment-free under design temperature and dead load, and hence plate thicknesses are in many areas controlled by minimum thickness requirements and relative deflections rather than stress.

Recently in the United States, the Carquinez Straits Suspension Bridge in Vallejo, California employed a 3.0 m (10 ft) deep, 27.2 m (89 ft) wide (cable-to-cable) orthotropic box girder as the stiffening element, weighing approximately 10,890 kg/m (1,506 lb/ft). For the Detroit River International Crossing, the box girder could be expected to weigh on the order of 13,540 kg/m (1,872 lb/ft), based on the increased span between cables.

#### **8.1.5.2. Stiffening Truss**

The stiffening truss has been used as the stiffening element in suspension bridges for over a hundred years. Traditional stiffening trusses had been fabricated with riveted and subsequently bolted construction. Modern stiffening trusses, however, employ fully welded construction, continuous trusses, integral orthotropic decks and field bolted connections where desirable. The integral orthotropic deck results in material efficiencies, allows for a joint-free deck from end-to-end of the structure thereby protecting the superstructure steel from roadway runoff, and offers a service life equal to that of the overall structure.

Currently, the Third Tacoma Narrows Bridge is being erected over the Puget Sound with a shop welded superstructure, an integral orthotropic deck, and bolted field joints as appropriate. The Tacoma Narrows truss is 7 m (23 ft) deep and 23.8 m (78 ft) wide between cables. For the Detroit River International Crossing, a stiffening truss approximately 7.5 m (25 ft) deep would be anticipated, with an approximate weight of 17,470 kg/m (2,415 lb/ft).

#### **8.1.5.3. Summary**

For any of the three crossings, both the orthotropic box girder and the stiffening truss are viable alternatives. For the purposes of this report, the orthotropic box girder will be estimated and developed, however it is recommended that the option of a stiffening

truss remain open for the bridge types discussed in **Section 2.4** and studied in further detail at the conceptual design phase of the project. Cost estimates for the stiffening truss and orthotropic box girder alternates are anticipated to be comparable at this level of study.

#### **8.1.6. Fabrication**

While wire, wire rope and structural strands for the suspension system may be procured at competitive prices from any one of a number of qualified suppliers around the globe, steel fabrication of the magnitude required for the superstructure is typically accomplished by offshore fabricators, as North American fabricators may no longer be cost-competitive for this sort of work on the world market. In fact, the Carquinez box girder and Tacoma Narrows truss were fabricated in Japan and South Korea, respectively, and transported across the Pacific Ocean on large ships.

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. Often structures of this magnitude are fabricated off-site into panels, transported to the site and panels assembled on-site or at a nearby assembly yard. This remains an option for this project. However, notwithstanding 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.

Major bridge structures also employ a significant amount of cast and forged steel components. Cable bands, strand shoes, rocker link components, etc. are examples of such components. Domestic and foreign foundries have historically been competitive supplying these components.

#### **8.1.7. Erection**

For economy, in recent years on-site erection of the towers and anchorages employ traditional cast-in-place concrete methods. Steel towers are typically no longer cost-competitive with reinforced concrete and maintenance painting during the life of the structure further detracting from the use of steel in these members. However, some schedule savings may be realized with the use of steel towers.

Because of high compressive loads imparted onto the tower legs, prestressing is not required in the vertical elements of the towers, however prestressing of the tower struts would be an advantage. The tower legs may be formed either with jump forms or by slipforming, though jump forms have become the more common method in recent years.

Due to the shear volume of concrete employed in gravity cable anchorages, care is required to monitor the heat of hydration within acceptable limits. This is typically accomplished through staging of the concrete pours, rather than specific measures to cool the concrete such as introducing ice into the concrete mix or circulating coolants through piping set in the curing concrete.

Once the towers and anchorages have been completed, hauling lines and a catwalk are erected from anchorage to anchorage, mimicking the profile of the cables. The catwalk serves as a working surface from which crews can perform their duties while spinning the main cables and erecting the suspension system.

With the cables spun and compacted, cable bands and subsequently the suspenders are erected. Lifting gantries are erected atop the cable that hoists the deck segments from a barge or directly from the ocean-going ship. The deck segments consist of a complete block of the bridge, i.e. all the structural steel for the entire width and predetermined length of the bridge, as well as miscellaneous pieces and equipment determined by the contractor. Utilizing progressive load transfers these segments may be 'trapezed' along the bridge and into position under existing structures in the side span, which can be particularly advantageous in areas where avoidance of surface features is important, such as the LaFarge rail line at X10 or Sterling Fuels at X11(C).

A significant difference between the suspension and cable-stayed bridge structure types is that the suspension bridge's stressless stiffening element allows for field splicing activity to be moved essentially off the critical path. Also, lighter deck segments and vertical support cabling allow prefabricated suspension bridge deck segments to be substantially larger than those for cable-stayed bridges, thereby reducing erection durations.

Site conditions at the proposed crossing locations do not pose any unusual difficulties that require specialized or high-risk operations. However, it should be noted that for locations without direct vertical access from a barge or land transport, some trapezing of segments – using progressive load transfers between inclined lifting lines to swing segments longitudinally along the length of the bridge – are necessary. This has become common practice.

## **8.2. Suspension Bridge Engineering Studies**

For the purposes of the Bridge Type Study, engineering studies have been limited to the identification of viable variations for the major bridge components and comparative studies of recent structures that provide insight into the magnitude and appropriateness of structural variations for the Detroit River International Crossing.

As the project moves into the Conceptual Design Phase, engineering studies will include preliminary analysis and member sizing, further development of the design criteria, and revised quantity takeoffs for the main crossing. It is anticipated that from these studies project estimates may be further refined.

## **9. Cable-Stayed Bridge Options**

### **9.1. Description of Cable-Stayed Bridge Options**

#### **9.1.1. Layout**

The span arrangement for the cable-stayed bridge options are three-span cable supported units with two pylons. Ideally, and where possible, the side-span to main-span ratio is set

at about 0.45. This historically has resulted in an efficient design with manageable uplift at the ends of the side spans. The bridge alignments have been established such that the three-span unit can maintain a tangent alignment. Where the alignment dictates a curvature at the end of the side span, some options have a side-span/main-span ratio as low as 0.35 in order to keep the curvature off of the cable supported side span. This will result in significant uplift forces in the side span, and will require intermediate side span piers, also known as auxiliary piers, and/or other measures to assist in resisting the uplift forces.

Span variations remain available to avoid possible obstructions on land or improve economy. For instance, side span lengths may be varied, or intermediate side span piers can be used. As the project moves forward, such refinements may be considered as a matter of economy or appropriateness for the site. Further details are covered in the individual bridge option evaluations.

### **9.1.2. Cable System**

In general, the stay cable arrangement can be described as one of three configurations: Harp, Fan or Semi-Fan. The Fan arrangement represents the most efficient usage of stay material, since the individual stays are as near-vertical as possible. However the concentration of a large number of stays at the top of the tower creates congestion difficulties, which are particularly problematic for long spans with a large numbers of stays. The Harp arrangement represents the least efficient usage of stay material but has an advantage that deck construction can begin well before the tower construction is completed. The Harp arrangement also requires intermediate piers in the side-span to improve the overall stability. The semi-fan represents a good compromise between minimizing congestion at the upper anchorage of the pylon while maximizing the inclination of the stays. This stage of the study will use the semi-fan arrangement for all options, however, this may be studied in further detail at the conceptual design phase of the project.

The spatial arrangement of the stay cables can in general be classified as the following:

- Vertical cables anchored in the median (one plane of stays).
- Vertical cables anchored at the edges of the deck (two planes of stays).
- Inclined cables anchored at the edges of the deck but converging at the top (two planes of stays) of the pylon.

Obviously the spatial arrangement of the stays, the pylon arrangement and the bridge deck-system (main girder) arrangement go hand-in-hand with one another.

The spatial arrangement of the cables has an influence on the structural behavior of the global bridge system in relation to how it carries torsional loads and its aerodynamic performance. Vertical cables either in the median or anchored at the edges have a minimal contribution to the torsional stiffness of the system. However, inclined cables anchored at the edges of the deck and joined at the tower top create essentially a “space-truss” with the deck, and impart significant torsional rigidity to the system. For the long-spans envisioned for this study and the corresponding importance of aerodynamics, the

inclined cable arrangement has been utilized to provide an added contribution to the torsional behavior of the system.

The stays themselves typically consist of 7-wire prestressing strands protected individually with wax and polyethylene sheathing. Epoxy coated or galvanized strand has also been used, however there are no domestic fabricators of galvanized strand. Individual stays are made up of multiple strands encased in a high density polyethylene pipe. The pipes are supplied in a wide range of UV-resistant colors. An outer helix bead can also be incorporated to mitigate against rain and wind induced vibrations.

The strands are anchored using wedges seated in and 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 also possible to remove and replace individual strands at any point in the life of the structure. This ease of stay replacement provides a future maintenance benefit over suspension cables.

### 9.1.3. Pylon

There have been a multitude of pylon arrangements that have been utilized for cable-stayed bridges, but they can be classified into five general types:

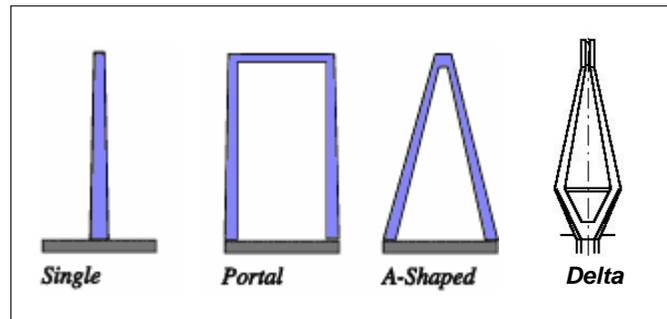
- Single column pylons in the median
- "H" pylon
- "A" pylon
- Delta pylon (or Diamond)
- Inverted "Y" (this can be used with either "A" or delta configuration)

The single column arrangement requires the deck to be bifurcated in order to pass the pylon between the roadways. This necessitates widening of the deck by the width of the pylon. Given the relatively wide deck for this project, this is not considered the most economical overall bridge arrangement. The "H", or "Portal", tower is characterized by vertical or near-vertical

orientation of the stay cable planes. There is an important consideration of this tower arrangement as related to its lack of contribution to the torsional stiffness of the deck.

With the portal tower arrangement and vertical stay cables a torsional rotation of

the deck is accompanied by the tops of the two tower legs moving in opposite direction. There is little resistance to this movement, and the tower/stay arrangement contributes relatively little to the torsional stiffness of the deck system. On the other hand, for the A-shaped or delta arrangement, a torsional rotation of the deck results in a small movement of the top of tower since the two legs are joined at the top. This results in a significant contribution to the torsional stiffness of the deck. For a relatively long cable stayed bridges



subjected to wind actions, it is considered that this additional torsional stiffness will be necessary to achieve an acceptable and economical design. Therefore we have chosen not to further consider a portal tower arrangement for the cable stayed bridge options. The "A" tower has the advantage of a wide foundation to resist overturning and eliminates the deviation of column leg forces just below deck level required with the delta tower. The inverted "Y" configuration has the advantage of grouping all of the upper stay anchorages in a compact arrangement, which is considered an advantage for constructability. And may provide a superior design. Therefore, the tower arrangement considered for options with land piers will be an "A" tower arrangement with an either inverted "Y" shape above deck or legs joined near the top of tower (a conventional "A" shape). The smaller foot print of a Delta pylon reduces marine construction costs and the potential for vessel collision, so the Delta pylon will be considered for options with river piers.

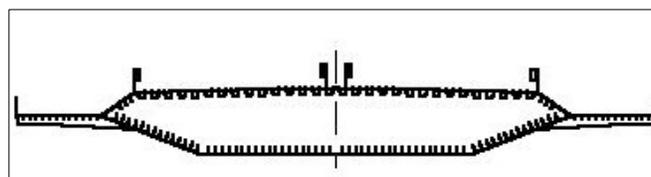
#### 9.1.4. Deck System

There are two basic material choices for the bridge deck system, concrete and steel. Although both materials have been used successfully for cable-stayed bridges of more moderate spans, steel is the obvious choice for very long spans such as those contemplated for the DRIC with the possible exception of Option 5. The primary reason is the significant dead load that must be carried by the stays, pylon and foundation. However, the use of a concrete side-span deck system for alternatives with shorter than optimum side-span lengths may be advantageous to provide sufficient ballast. For the purposes of this phase of the study a consistent deck system is assumed for the full length of the bridge.

There are two basic deck systems that can be considered for a cable-stayed bridge, an "open" system comprised of edge girders and floor beams or a "closed" system with a box girder arrangement. A fundamental difference in the two systems is that the open system is relatively weak torsionally and relies on the cables to provide most of the torsional restraint of the system. The "closed" box girder system is relatively stiff torsionally.

##### 9.1.4.1. Box Girder

The geometry of the stay cables of a cable-stayed bridge are such that significant compressive forces are imparted on the deck elements. Under these conditions, the



*Example Orthotropic Box Girder Deck System*

design of the box girder must consider local plate buckling. Orthotropic box girders are typically only economical for very long spans where minimum superstructure weights are necessary and

aerodynamics are critical. Box girders for short and moderate span cable-stayed bridges are typically precast or cast-in-place prestressed concrete similar to a segmental box girder structure. However, a concrete superstructure is not cost effective for the spans being considered for this project. An orthotropic girder weighing

the same order of magnitude as a suspension bridge would likely be an economical solution for the span ranges under consideration for this study.

#### **9.1.4.2. Edge Girder**

Edge girder deck systems consist of heavy girders, either concrete or steel, along the plane of the cables on either side of the deck. Floorbeams span transversely between these edge girders with a concrete deck spanning longitudinally between floorbeams. A composite steel system is probably not feasible for spans over 600 m (1,970 ft) due to their poor aerodynamic performance and the significant dead load of the deck.



#### **9.1.4.3. Summary**

The aerodynamic stability tends to dominate the structural performance of both the final and erection stage conditions. There will be an advantage to the box girder configurations, both from the torsional rigidity provided by the closed cross section and by the aerodynamic shape that can be imparted into the cross section. Given the span requirements for this project, an orthotropic steel box girder configuration will be used for purposes of the Type Study.

### **9.1.5. Fabrication and Erection**

The basic erection method most appropriate for the cable-stayed bridge alternatives is a balanced cantilever erection method. The basic bridge construction sequence will be to construct foundations, pylons and piers, then erect a pier table to begin the deck construction. The superstructure sections will be brought to site in sections 9 to 15 m long and lifted to deck level with either mobile cranes or stiff leg derricks placed on the deck. As each deck section is installed, the corresponding pair of cables is installed. The deck sections are alternately erected on the main span and side span, such that they never get more than one section out of balance. The deck cantilevers out from each tower until side span closure is reached, and then main span closure

It is of note that cable-stayed structures are restricted to vertical lifts when lifting deck segments. For the DRIC, this may present some challenges due to the development of sites over which the sidespans traverse. This may be overcome by utilizing cast-in-place construction, temporary trestles, or additional land acquisition.

Unlike the suspension bridge alternatives, the configuration of a cable-stayed bridge imparts large compressive loads through the deck element. This requires field splicing to be an integral part of the lifting cycle and hence comprises part of the construction schedule critical path. With this erection sequence, the erection stresses are in general kept within the envelope of final service stresses and therefore do not govern the overall design. The next phase of the study will examine the efficacy and cost of supplemental

side span temporary piers (for layouts that do not already have additional side span piers) to assist in reducing aerodynamic buffeting effects during erection.

It will be assumed that steel deck sections will be fabricated as full width sections with the length equal to the stay spacing, and delivered by barge, so no over-road size restrictions apply.

## **9.2 Cable-Stayed Bridge Engineering Studies**

For the purposes of this study, cable-stayed alternatives have been developed for each crossing. This includes alternate crossing conditions for a pylon in the Detroit River or clear spanning the river, as appropriate. The alternatives were proportioned based on a combination of some basic ratios generally accepted for economical cable-stayed bridge construction (such as economical pylon height to span ratios) and based on comparisons with completed bridges of similar size. These proportions were then modified as needed for the specific geometric alignments.

The engineering studies at this stage have been necessarily limited in scope. They have focused on identifying the major bridge components that are most important in influencing the overall cost and then using a combination of preliminary calculations for sizing and comparative studies of recent structures to provide an estimate of the component sizing and costing.

As the project moves into the Conceptual Design Phase, more detailed engineering studies will include preliminary analysis and member sizing, further development of the design criteria, and revised quantity takeoffs for the main crossing. At the Conceptual Design stage alternative structure configurations will be explored to examine the economy of differing structural arrangements. It is anticipated that from these studies project estimates may be further refined.

## **10. Crossing X10(A)**

### **10.1. Main Bridge Types**

Given the length required to clear span the river on this alignment only one suspension bridge option has been considered to span from shore to shore, additionally, a suspension and cable-stayed option have been developed considering a river pier on the Canadian side of the navigation envelope. If a river pier or piers are allowed, a cable-stayed bridge may be considered, although it would be at the upper envelope of current cable-stayed construction.

### **10.2. Span Arrangements**

Three options have been considered at Crossing X10(A). Regardless of the option, this alternative requires a highly skewed crossing (approximately 49° between a line perpendicular to the centerline of channel and the centerline of the bridge alignment), and very large main spans.

### **10.2.1. Type Study Option 1**

Type Study Option 1 is a suspension bridge that spans crossing X10(A) with a single 1,300 m (4,265 ft) main span (the largest main span of all options considered) with both towers on land and two unsuspended side spans. Although suspension bridges with main spans far exceeding 1,300 m exist, at this span length this would become the longest span bridge in the Americas, edging out the Verrazano Narrows and Golden Gate bridges by 1.6 m and 19.9 m, respectively.

### **10.2.2. Type Study Option 2**

Type Study Option 2 is a suspension bridge that shortens the length of the main span by placing one pier on the Canadian side of the river. With this configuration, the main span is 925 m (3,035 ft). While shortening the main span with respect to Type Study Option 1 yields certain cost savings, these savings are offset to some degree by pier protection for the marine tower, as well as the cost of suspending the side span adjacent to the river pier. The Canadian tower is in the Detroit River and therefore requires protection from vessel collision forces and other river loadings. The U.S. tower and anchorages are all on the land. The entire cable supported unit is on a tangent alignment.

### **10.2.3. Type Study Option 3**

Option 3 is a three-span symmetric cable-stayed bridge with a 925 m (3,035 ft) main span. This option represents the longest of the cable-stayed options that are considered for this project, would be the 3rd longest cable-stayed span in the world, and the longest in North America by a significant margin. The side spans are set at 416 m (1,365 ft), giving a side span/main span ratio of 0.45. Due to the very long main span, the Conceptual Design phase may consider concrete side spans, additional side span anchor piers or other means to counteract live load and dead load imbalances. The tower on the Canadian side is placed in the Detroit River and will be protected with dolphins. The U.S. tower and anchor piers are all on land. The entire cable supported unit is on a tangent alignment.

## **10.3. Approaches**

The U.S. approach to the main span is on structure between the U.S. abutment and the U.S. anchorage/anchor pier or main pier, the Canadian approach is on structure from the Canadian anchorage/anchor pier or main pier to the Canadian abutment. The abutment locations are based on an abutment height of 3 m (10 ft) and the location depends on the vertical profiles developed for the different Type Study options. The total length of approach depends on the main span length, the use of a suspended or unsuspended side span, the existence of a river pier and the associated vertical profile.

**Table 8** details the number of the approach span and the total length of the approaches. During the next phase, Conceptual Design, structure types and optimum span lengths will be examined, recognizing possible differences between US and Canadian industries.

*Table 8. Length of Crossing X10(A) Approach Structures.*

TS Option	US Approach – m (ft)	CAN Approach – m (ft)	Total – m (ft)
1	929 (3048)	1771 (5810)	2700 (8858)
2*	901 (2956)	1771 (5810)	2672 (8766)
3*	485 (1591)	1730 (5676)	2215 (7267)

\*Canadian side river pier.

## 11. Crossing X10(B)

### 11.1. Main Bridge Types

On this crossing alignment both suspension and cable-stayed bridges are under consideration. In addition, the option of placing a pier or piers in the river is under consideration. For suspension bridges both suspended and unsuspended side span layouts have been developed.

### 11.2. Span Arrangements

Five crossing alternatives have been advanced at crossing X10(B), two cable-stayed options and three suspension bridge options. For options having piers in the river, the navigation channel is shifted either east or west to minimize the required span lengths. This channel shift also results in a relative change in the skew angle (the angle between a line perpendicular to the theoretical centerline of channel and the centerline of the bridge alignment), varying from 8 degrees to 25 degrees.

#### 11.2.1 Type Study Option 4

Option 4 is a three-span symmetric cable-stayed bridge with an 860 m (2,822 ft) main span. Side spans are set at 300 m (984 ft) in order to avoid introducing curvature into the side spans. For this option, both pylons and all anchors are out of the water and not subject to vessel collision loading. This arrangement gives a side span/main span ratio of 0.35 which will result in significant uplift at the side span pier locations. In order to assist with the mitigation of this effect, auxiliary side span piers and ballast have been included to help distribute the uplift forces. These side span piers will also function to stiffen the structure for live load deflections for this very long main span, and will contribute to stiffening the structure during the erection stage in response to wind and erection loadings.

#### 11.2.2. Type Study Option 5

Type Study Option 5 is a cable-stayed bridge which spans crossing X10(B) with the shortest main span length, along with Option 8, of all Type Study options by shifting the navigation channel towards the US river bank and placing one main pier in the river on the Canadian side. The resulting main span length is 600 m (1,969 ft). The 282 m (925 ft) side spans are entirely supported by the stay cable system. Additional navigation through the side span next to the river pier is feasible and will be further investigated. The main pier shape will likely be delta shaped with the tower legs coming together below the deck to

minimize the foundation footprint in the river. Additional economies may be realized during the Conceptual Design phase by considering an edge girder deck system for this relatively short main span.

### **11.2.3. Type Study Option 6**

Type Study Option 6 consists of an 870 m (2,854 ft) main span suspension bridge with both towers on land and suspended side spans. The main span length of this structure is comparable to the Tacoma Narrows Bridge currently being constructed at 853 m (2,800 ft).

### **11.2.4. Type Study Option 7**

Type Study Option 7 is a suspension bridge identical to Option 6 above, except that in this option the side spans are not suspended. This option presents an economical solution with little construction risk as all operations are land based and provides the maximum navigation channel.

### **11.2.5. Type Study Option 8**

Type Study Option 8 is a suspension bridge which shortens the length of the main span by placing one pier in the river on the Canadian side. With this configuration, the main span is 600 m (1,970 ft), the same as for Option 5. While shortening the main span with respect to Type Study Option 7 yields certain cost savings, these savings are offset to some degree by the necessary pier protection for the marine tower, as well as the cost of suspending the side spans adjacent to the river pier, and additional construction and mitigation costs for in-river work. Additional savings may be realized during the Conceptual Design phase by modifying the U.S. side span to an unsuspended configuration.

## **11.3. Approaches**

The U.S. approach to the main span is on structure between the U.S. abutment and the U.S. anchor or main pier and Canadian approach from the Canadian anchor or main pier to the Canadian abutment. The abutment locations are based on an abutment height of 3 m (10 ft) and the location depends on the vertical profiles employed for the different Type Study options. The total length of approach depends on the main span length, use of a suspended or unsuspended side span, the existence of a river pier and the associated vertical profile.

**Table 9** details the total length of the approaches. During the next phase, Conceptual Design, structure types and optimum span lengths will be examined recognizing possible differences between US and Canadian industries.

*Table 9. Length of Crossing X10(B) Approach Structures*

TS Option	US Approach – m (ft)	CAN Approach – m (ft)	Total – m (ft)
4	637 (2090)	387 (1270)	1024 (3360)
5*	631 (2070)	564 (1522)	1195 (3592)
6	832 (2730)	402 (1320)	1234 (4050)
7	1022 (3353)	592 (1942)	1614 (5295)
8*	998 (3274)	269 (883)	1267 (4157)

\*Canadian side river pier.

## 12. Crossing X11(C)

### 12.1. Main Bridge Types

At this crossing location both cable-stayed and suspension bridge options are under consideration. Piers in the River are not being considered for this crossing alternative because the required horizontal navigation clearance of 526 meters is nearly the full width of the river at this location, and any minor savings by slightly shortening the spans would be more than offset by the costs necessary to protect the piers from an errant vessel impact

### 12.2. Span Arrangements

The width of the navigation channel at crossing X11(C) is nearly as wide as the river at this location and any cost savings associated with a reduction in span length would be offset by additional construction and mitigation costs.. Therefore, all three crossing alternatives that are being considered at this location clear span the river. The proposed alignment is on a skew angle of approximately 29° (between a line perpendicular to the centerline of channel and the centerline of the bridge alignment) with a main span length of 750 m (2,461 ft) for all crossing options.

#### 12.2.1. Type Study Option 9

Type Study Option 9 is a 750 m (2,461 ft) cable-stayed structure with land based pylons adjacent to the river banks. The 363 m (1,191 ft) long cable supported side spans feature two anchor piers each. On the U.S. side the anchor piers are spaced to clear Jefferson Avenue. The Canadian side span layout is symmetrical to the U.S. side span.

The inverted Y shaped pylons stand 170 m (558 ft) above the profile grade line for a total pylon height of 210 m (689 ft).

#### 12.2.2. Type Study Option 10

As a result of the horizontal alignment as discussed above, Type Study Option 10 features a suspension bridge with a 750 m (2,461 ft) main span, two land-based piers and unsuspended side spans.

### 12.2.3. Type Study Option 11

Type Study Option 11 is a suspension bridge identical to the previous option, except that 250 m (820 ft) suspended side spans have been considered. Unless certain extenuating circumstances not currently evident were to come into play (the discovery of a brine well that needed to be bridged, the discovery of contaminated soils that would be unreasonably costly to mitigate, etc.), this option would not be economically justifiable if Option 10 were to remain viable.

## 12.3. Approaches

The U.S. approach to the main span is on structure between the U.S. abutment and the first U.S. anchor or main pier. The abutment location is based on an abutment height of 3 m (10 ft) and its location depends on the vertical profiles employed for the different Type Study options. On the Canadian side three alternate approach alignments are being investigated. The first alignment connects the main bridge with Canadian Plaza Option A, the second alignment routes traffic to Canadian Plaza Option B, while the third alignment connects to Canadian Plaza C. An elevated alignment is maintained between the main bridge and the plaza for security reasons. Table 10 details the total length of the approaches. During the next phase, Conceptual Design, structure types and optimum span lengths will be examined recognizing possible differences between US and Canadian industries. Also, modifications to the horizontal alignment to avoid the Keith Transformer station will be developed.

*Table 10. Length of Crossing X11(C) Approach Structures*

TS Option	To Canadian Plaza	US Approach – m (ft)	CAN Approach – m (ft)	Total – m (ft)
9	B	391 (1283)	1151 (3776)	1542 (5059)
9	C	391 (1283)	956 (3136)	1347 (4419)
10	B	785 (2575)	1514 (4967)	2299 (7542)
10	C	785 (2575)	1316 (4318)	2101 (6893)
11	B	498 (1634)	1270 (4167)	1768 (5801)
11	C	498 (1634)	1075 (3527)	1573 (5161)

## 13. Comparative Construction Cost Estimates

### 13.1. Methodology

The cost estimates are in 2006 US dollars and do not include soft costs such as inflation, property acquisition, or final and construction engineering. A comparative quantity based estimate methodology was used to determine relative costs of the crossing alternatives. This estimating methodology used structures of similar type and scale in the U.S. and Europe (such as Pont du Normandie, Carquinez Straits, Tacoma Narrows, etc.) then scaled the gross structural dimensions for each type study option for an approximation of the quantities.

This estimating approach only considered the major structural components such as the anchorages/anchor piers, tower/pylon, foundations, superstructure, pier protection, and suspension system. Limited structural design was performed at this stage. Unquantified items, such as deck overlay, lighting, drainage, appurtenances, etc. were calculated using per square meter cost established by using the same comparative approximation method. The estimate summary can be found in **Appendix C: Cost Estimate Summary**.

The resulting square meter cost for the alternatives was then plotted on an historical unit cost graph, **Figure 9**, to check the reasonableness of the estimates. This verification process served to confirm the order of magnitude of the estimates.

Finally, the contingency amounts were varied to reflect the real variability in project costs. By providing the costs as a range versus a single number helps the public to better understand that the project is at a very preliminary stage.

### ***Contingencies***

Contingencies are apportioned to reflect the risks and uncertainties relative to each option at this stage of project development. Specifically, the *design contingency* is established relative to the amount of design work performed and the degree of comfort with the current level of design. For the main river bridge the costs were developed based on a significant amount of historical data augmented with limited design work. This approach provided the best confidence level since the level of design effort is necessarily limited at this stage and the use of existing bridge cost data, with appropriate adjustments for inflation to current year and for geographic differences. Additionally, the project team had more suspension bridge data in the structure size ranges being considered than cable-stayed data. Therefore, the design contingency was established between a range of 10 to 20%, with the lower range used for the suspension bridge options and the higher range used for the cable stayed bridge options. For the approach bridges the estimates were based only on historical costs per square meter with no work done on span optimization or the bridge interface with the plaza. The contingency for this work was set at 25%.

For the *construction contingency*, which reflects uncertainties with regard to cost volatility and unforeseen items, a cost sensitivity analysis was performed which examined the effects of unit cost volatility on the major quantities such as structural steel and concrete. This analysis found that a maximum 20% construction contingency was warranted.

Other project contingencies, such as a *management contingency*, for third party changes, environmental mitigation, or changes in the project scope, were not included in the bridge estimates but may be incorporated into the overall project cost as is appropriate.

## **13.2. Unit Costs**

At this stage of most projects, historical unit costs are used to estimate the bridge costs. An analysis of area unit costs, derived from an historical survey of long span bridges constructed in the U.S. in the past 25 years, was performed. These costs were then adjusted using RS Means geographical index and ENR's construction year index for 2006 US dollars. The resulting graph is shown in **Figure 8**. A regression analysis was then performed resulting in a

unit cost equation based on main span length which yields a cost formula based on a bridge's main span length.

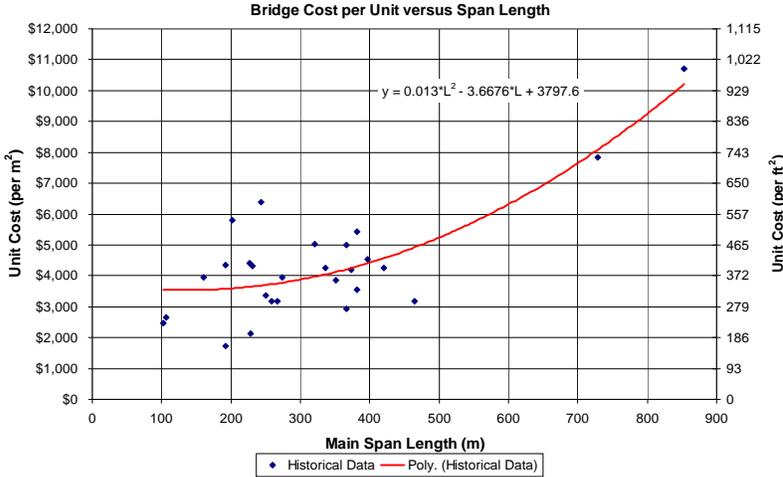


Figure 8. Historical bridge unit costs versus span length.

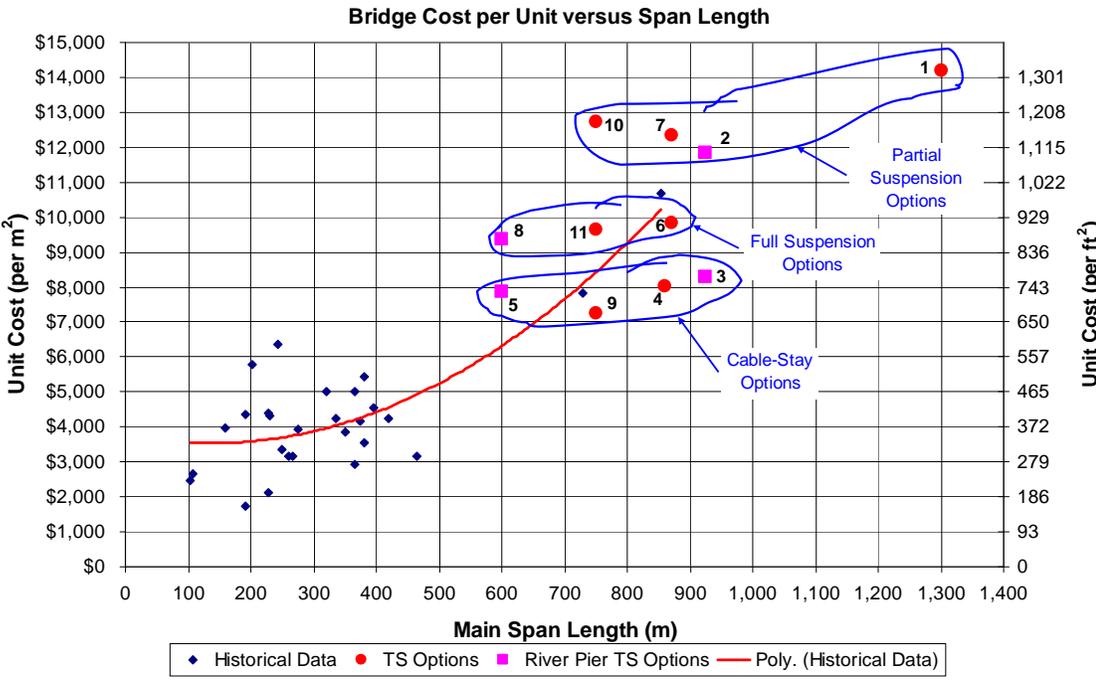


Figure 9. Historical unit costs and Type Study option unit cost estimates.

Approach span costs were based on an average cost per square meter for spans in the range of 45 to 60 m (148 to 197 ft). The costs were developed using an average of current costs for similar bridges in Michigan and Ontario, adjusted to 2006 US dollars for consistency with the main bridge costs. During the Conceptual Design phase these costs will be refined to reflect

consideration of different structure types as well as differences in methodologies and market costs between the US and Canadian sides of the structure.

### **13.3. Construction Cost Risks**

The comparative cost estimates provided in the following section assume present day amounts for labor and materials. The final constructed cost may be subject to several factors. These may include volatility in material costs, labor shortages, unanticipated subsurface conditions, difficulties with marine construction for those options with marine elements, etc. At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 representing the least risk.

#### **13.3.1. Material Cost Volatility**

In recent years, high demand from overseas consumers and hurricane related reconstruction efforts in the US has driven up the cost of construction materials (steel, concrete, reinforcing steel) significantly. This consumption has cooled in recent months and may continue over the near future, with some declines projected for the current year. This source of cost risk is difficult to predict or mitigate.

#### **13.3.2. Labor Shortages**

The current construction market has been very aggressive in recent years. Coupled with an aging construction industry, labor has been in short supply in many parts of the United States and Canada. Whether or not the construction market demands may be reduced during the life of the project is unknown.

#### **13.3.3. Unanticipated Subsurface Conditions**

The Detroit River International Crossing has an advantage regarding subsurface features due to an aggressive and thorough geotechnical investigation prior to the design phase. While this work is focused on the identification and delineation of brine wells, data is also being collected that will aid in mitigating risk exposure to unknown subsurface conditions.

Given the industrial history of the site, it should be anticipated that some level of contaminated soil will be encountered. If these areas are identified early in the project, costs may be mitigated by bridging over them, or by other means. If these areas were to be discovered during construction, cost and schedule impacts could be significant.

#### **13.3.4. Marine Construction**

For those options with towers in the water, (Options 2, 3, 5 and 8) certain risks are assumed that do not apply for land-based construction. These include a construction season that may be limited due to ice flows, protection of marine habitats, limited access, more stringent environmental compliance, additional permitting, coordination with marine navigation, etc. Although not a cost risk per se, marine piers would also be protected by dolphins or other features to prohibit ship impacts, which would have a significant associated cost. This has been reflected in the Comparative Cost Estimates section of this report.

### **13.3.5. Structural complexity**

With the exception of the very long cable-stayed options presented in Options 3 and 4, the span lengths, cross sections and configurations presented have been well represented in the world's bridges, and hence are not anticipated to provide undue technical challenges.

### **13.3.6. Buy America**

As discussed previously, recent major steel spans have typically been fabricated overseas. The Carquinez Straits was fabricated in Japan, Tacoma Narrows in South Korea, and the Oakland Bay Bridge is currently contracted to be fabricated in China. These recent projects demonstrate that U.S. fabricators are more costly than overseas suppliers. Introducing a Buy America clause would undoubtedly increase the cost of a steel superstructure as compared to overseas procurement. These costs are not included in the cost estimates developed for this report, this factor will be given additional consideration in the Conceptual Design phase.

### **13.3.7. Contractor Availability**

Similar to the labor shortages discussed above, contractor availability has increased project costs in recent years. Additionally, certain features of the project lend themselves to a limited number of contractors. For example, the towers for Option 3 be nearly 300 m (1,000 ft) in height which would be around 80 m taller than the Renaissance Center, Detroit's tallest building. Structures of this magnitude may limit the number of competitive bidders.

## **13.4. Comparative Cost Estimates**

The following table summarizes the cost estimates for the Detroit River crossing from abutment to abutment. The construction estimates are in 2006 U.S. dollars and do not include soft costs such as engineering or inflation. This report does not consider the division of costs between the parties who will fund and execute the construction.

At this stage of development the cost estimates should only be used for comparison purposes and should not be used for project programming.

Table 11. Construction Cost Estimate Range.

Crossing	Type Study Option	Bridge Type	River Pier	Construction Cost Estimate – 2006 US\$ (000,000's)
X10(A)	Option 1	Susp.	N	770 - 920
	Option 2	Susp.	Y	680 - 810
	Option 3	CS	Y	620 - 740
X10(B)	Option 4	CS	N	430 - 510
	Option 5	CS	Y	370 - 440
	Option 6	Susp.	N	480 - 550
	Option 7	Susp.	N	470 - 540
	Option 8	Susp.	Y	420 - 490
X11(C)	Option 9	CS	N	450 - 530
	Option 10	Susp.	N	500 - 580
	Option 11	Susp.	N	520 - 600

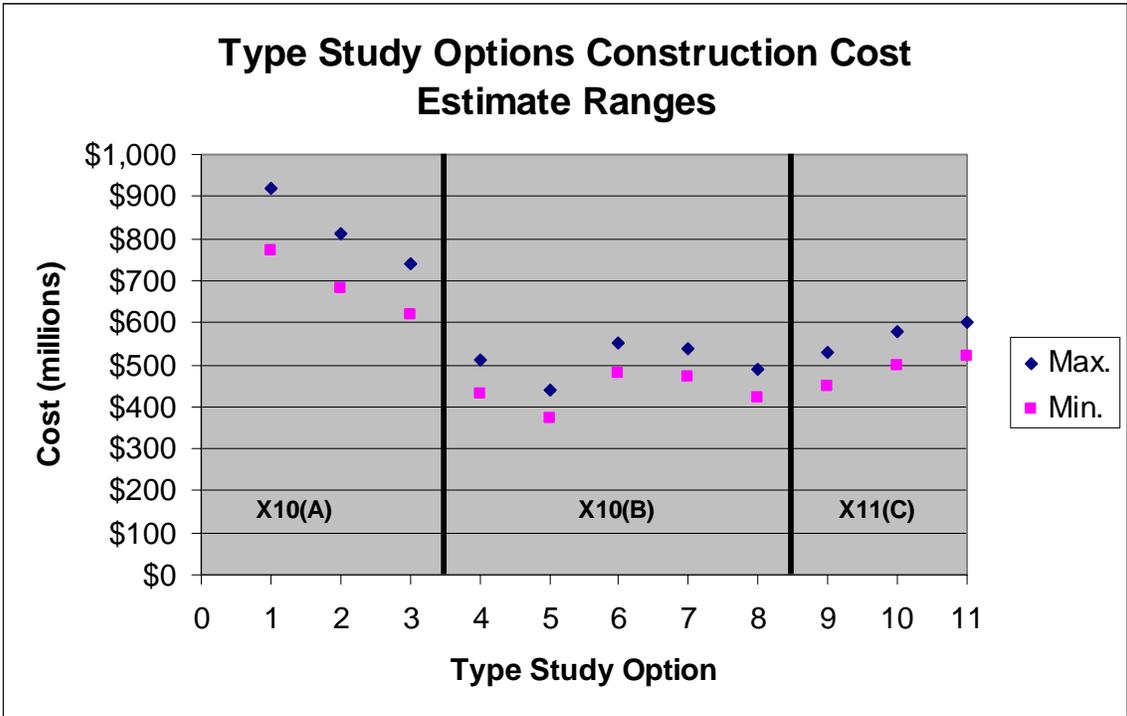


Figure 10. Construction Cost Estimate Graph.

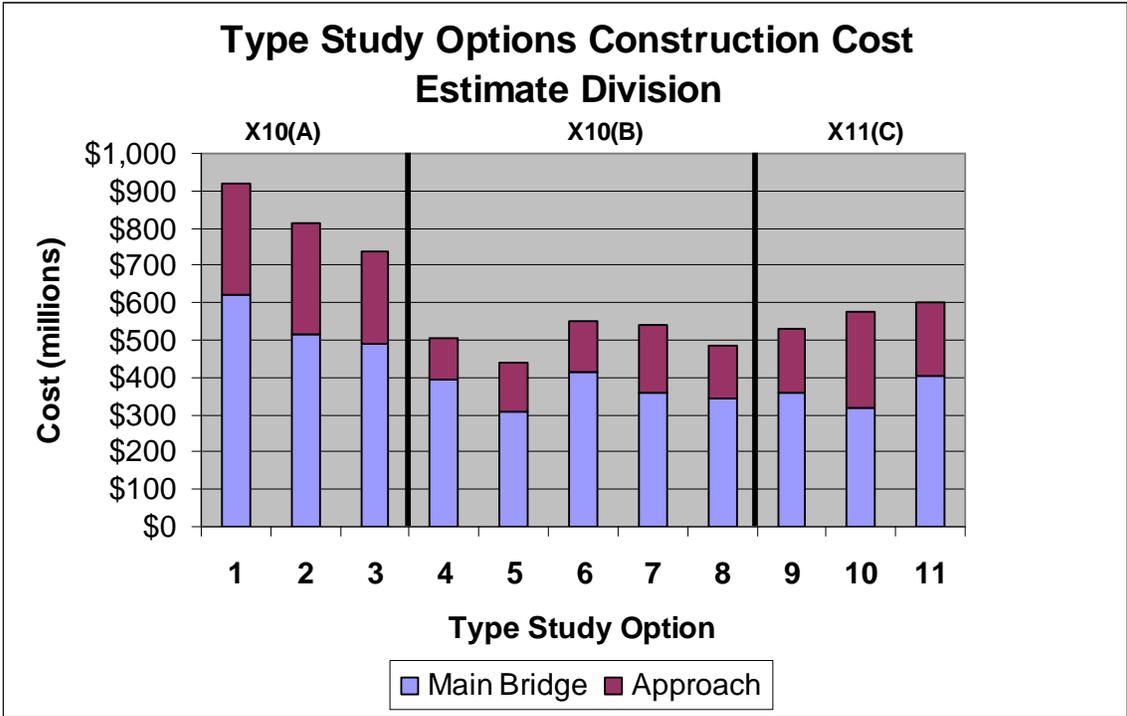


Figure 11. Construction Cost Estimate (Max.) Approach/Main Span Division Graph.

## 14. Constructability

### 14.1. Construction Schedule

Construction schedules were developed for each option using a consistent set of production factors [5] tied to the scale of the structure. These factors were developed using historical and industry standard data. The total schedule durations are shown in Table 12 while representative schedules may be found in Appendix D: Representative Construction Schedules.

For any option it is possible to accelerate the schedule using a variety of methods, however, acceleration will have an associated construction cost. Only a finite amount of schedule acceleration can be practicably achieved though, due to the scale of the structures and the linearity of the many critical path tasks. In some instances, such as the procurement of wire for a suspension bridge, it may be necessary to preorder materials in order to meet the schedule shown here or an accelerated schedule.

*Table 12. Estimated Construction Durations.*

Type Study Option	Duration (months)
<b>Crossing X10(A)</b>	
Option 1	62
Option 2	55
Option 3	57
<b>Crossing X10(B)</b>	
Option 4	52
Option 5	43
Option 6	49
Option 7	49
Options8	43
<b>Crossing X11(C)</b>	
Options 9	42
Option 10	51
Option 11	43

## **14.2. Construction Schedule Risk**

The overall construction duration of the project may be affected by a number of events, including geotechnical and marine setbacks, weather, labor strikes, material availability, utility relocation, site constraints, etc. This section will discuss the general risks to the construction schedule. At the Type Study phase this will be evaluated for each Option using professional judgment on a scale of 1 to 5, with 5 representing the least risk. It is presumed that permitting, land acquisition and other factors will have been addressed prior to the start of construction and accordingly these factors will not affect the construction durations.

### **14.2.1. Geotechnical Schedule Impact**

Unanticipated subsurface features, particularly mitigation of contaminated material or the presence of large boulders, could cause schedule slip. As discussed above, an aggressive front-end geotechnical investigation is an effective means to mitigate these risks.

### **14.2.2. Marine Schedule Impact**

For the reasons cited above regarding construction cost risk, it is again worth noting the inherent schedule risk associated with marine construction. Some of the cost savings gained from shortening the main span by utilizing river piers are offset by additional costs, cost risks and schedule risks associated with marine construction.

### **14.2.3. Inclement Weather**

Contract provisions for schedule adjustment with regard to inclement weather are typically included in construction contracts, and it is expected that such provisions will become a part of this project. The differential risk of delays to inclement weather with regard to the

different bridge options are of particular interest for this bridge type study. The principal difference in the alternatives is judged to be associated with marine schedule impacts for the options that have piers in the River, and the inclement weather related differential impacts are included as part of the Marine Schedule Impact above.

#### **14.2.4. Labor Strikes**

The threat of a labor strike is always of concern on a large project such as this. Hence, it is common practice for the contractor to reach a Project Labor Agreement with the local unions to prevent or lessen the likelihood of a strike.

#### **14.2.5 Material Availability**

Although material availability is always a concern for a project of this magnitude, these risks can be mitigated with adequate up-front planning and contracting. Also, as discussed above, overseas construction of construction materials has slowed, easing the global production strain.

#### **14.2.6. Utility Relocation**

Due to the location of the crossing in urban areas, a significant amount of utility relocations can be anticipated. As part of the project the major utilities, such as sewerage, natural gas, electrical, have been identified. The extent to which this will affect the schedule is not yet known and is beyond the scope of this report. Specific utility issues are discussed in Section 14.5.

### **14.3. Disruption to Local Users**

Construction of any of the bridge options will present a disruption to users of the local roadway system in the U.S. and Canada. However, this is not expected to be a major impediment. In some cases temporary detour routes may need to be designated. One area of concern is the potential for Sterling Marine to remain in operation during construction. The pipelines to the refueling dock present challenges to be overcome during construction for bringing superstructure segments to the site. At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 being least disruptive.

### **14.4. Construction Risk**

Construction risks include those factors that affect the bridge contractors' ability to price the work with a reasonable level of confidence. Some of these factors are site dependent, some are a function of the chosen general design solution and some will be dependent on the details of the final design and contract documents. Among those that can be identified now and that may be a factor or consideration for bridge type are the following:

#### **14.4.1. Towers in the River**

Those options that have a tower located in the Detroit River present additional construction risk due to additional hazards, exposures and delays related to working on the water, the possibility of a vessel collision with the construction works, increased exposure to weather delays, added exposure to environmental control violations and more difficult foundation construction requirements.

#### **14.4.2. Schedule**

All of the proposed alternatives are anticipated to be constructible within the time frame permitted by the overall project goals. Some of the longer span structures will obviously present more of a challenge in that their construction duration is nominally longer and it should be recognized that as the spans get longer, there is some level of inherent higher risk of schedule delay.

#### **14.4.3. Long Spans**

All of the proposed alternatives are within the span limits of other bridges that have been constructed elsewhere in the world, although some alternatives have spans near limits of record span length. This does not imply that the construction is routine, as each long span structure has its own unique construction requirements and challenges. In general the longer spans within each bridge type (cable-stayed and suspension) should represent increasing construction risk with increasing span, and between the cable-stayed and suspension bridge, the cable-stayed type presents a somewhat higher construction risk for the same span length.

#### **14.4.4. Construction Cost**

Even for routine construction projects there are risks involved in establishing an engineer's estimate for the project that will estimate the bidding. This risk is greater for long span bridge projects that are near the limits of record span length; also, the current bidding environment is such that where the construction cost trends have outpaced normal inflation. The estimated bridge costs include separate contingencies to account for unknown or un-quantifiable cost increases, but in the Conceptual Design phase there will also be a separate qualitative evaluation of the cost risk for the different options, with increasing cost overrun risk with increasing span length.

#### **14.4.5. Construction Experience**

It is noted that few bridges of the span lengths being considered have been constructed in North America in the past 40 or so years. This does not, however, mean that North American Contractors do not have this experience. Many North American contractors continue to participate in long span bridge construction worldwide. Additionally, it would be expected that a project of this magnitude will attract international contractors that will have experience in these bridge types. Therefore, the required construction experience for a major long span bridge is not viewed as a significant cost risk issue.

At the Type Study phase construction cost risk this will be evaluated using professional judgment on a scale of 1 to 5, with 5 representing the least risk.

### **14.5. Presence of Major Utilities**

At the Type Study phase this will be evaluated by counting the number of major utilities occurring at each crossing location, in both the U.S. and Canada, and then evaluating the relative risk these present to the constructability of the crossing using professional judgment on a scale of 1 to 5.

### **14.5.1. Major Utilities in U.S.**

The U.S. side of the project area is a major urban industrial area with many significant utilities transecting the area. These utilities include natural gas, electrical, and sewerage. At the Type Study phase this will be evaluated by counting the number of major utilities requiring relocation.

#### **14.5.1.1. X10(A) and X10(B)**

In the U.S. both alignments X10(A) and X10(B) land in the same vicinity. Just to the northeast of this location two high pressure natural gas transmission mains cross the river and continue through Delray. Adjacent to the Delray Boat Launch aerial electrical transmission lines cross the river and feed into the DTE substation on Jefferson Ave. Those transmission lines then cross Jefferson into Delray and Southwest Detroit. Along Jefferson Ave. a large diameter sewerage main feeds the City of Detroit sewerage plant on the west end of Delray.

The natural gas transmission lines and the sewerage line may be avoided by careful layout of the bridge substructure. It is likely that the electrical transmission lines would either need to be relocated or placed under ground, the latter option having some benefits from the standpoint of urban beautification.

#### **14.5.1.2. X11(C)**

In this location there is a major sewerage outfall approximately midway between the Fort Wayne property and the Mistersky Power Plant. The current alignment of the bridge avoids this outfall, however, the City of Detroit Water and Sewerage Department is planning a large Combined Sewer Outflow (CSO) retention basin in this area. There is sufficient area in this location to either modify the bridge alignment or adjust the location of the retention basin during design. This will require close coordination with the City of Detroit Water and Sewerage Department.

As is the case with X10, a large diameter sewerage main travels along the Jefferson Ave. right-of-way. This may be avoided by careful layout of the bridge substructure.

### **14.5.2. Major Utilities in Canada**

Utility impacts associated with all crossing alternatives which include hydro (electrical) transmission lines, the Lou Romano outtake, a high pressure gas pipeline owned by Dome Petroleum (and operated by British Petroleum) and the steam tunnel may be avoided by careful layout of the bridge substructure.

#### **14.5.2.1. X10(A) and X11(B)**

Crossing X10(A) will require some property acquisition from the Brighton Beach Power Station. Although the property requirements will not require relocation of any utility plant, the close proximity of exhaust stacks could result in potential visibility (steam) and odor concerns. Crossing X10(A) will impact a steam tunnel which runs below Sandwich Street, however, no other major utility impacts are associated with this crossing.

Crossing X10(B) will impact the northeast corner of the Keith Transformer Station. Although the alignment for Crossing X10(B) will not directly impact any of the main components of the transformer station, additional discussions with Hydro One staff are required to confirm that Keith Transformer Station can coexist with Crossing X10(B). This alternative will require modifications or relocation of three transmission lines which terminate at the Keith Transformer Station. There is a steam tunnel which runs below Sandwich Street which will also be impacted by Crossing X10(B).

#### **14.5.2.2. X11(C)**

Crossing X11(C) impacts two sets of transmission lines, outtake to the Lou Romano Reclamation Plant, the high pressure gas pipeline and the West Windsor Power Plant steam tunnel.

### **14.6. Presence of Contamination**

The bridge is located in heavily industrialized areas where contamination may be present. The presence of soil contamination on the project site would require some remediation. At the Type Study phase this will be evaluated by counting the number of contaminated sites registered with the appropriate agencies occurring at each crossing location, in both the U.S. and Canada, and then evaluating the relative risk these present to the constructability of the crossing using professional judgment on a scale of 1 to 5.

#### **14.6.1. Contaminated Sites in U.S.**

Environmental issues will likely be present for any excavations along the U.S. shoreline and within the upper 2 to 3 m (5 to 10 ft) of river sediment. Along the shoreline, fill soils to depths of 2 to 9 m (5 to 30 ft) from previous activity are typically contaminated requiring disposal in Type II landfills.

##### **14.6.1.1. Crossing X10 (Former Solvay – Detroit Coke Site)**

Both X10 crossing alignments land in the former Detroit Coke Site, originally owned by the Solvay Processing Company (Solvay), which occupies most of the area between Jefferson Avenue and the Detroit River. Due to the presence of regulated deep underground injection wells in the western part of the property, it was identified as a Resource Conservation and Recovery Act (RCRA) facility. Associated environmental impacts with the coke oven operations and coke oven gas by-products included tar, free phase hydrocarbons (free product), and soil and groundwater contamination. Almost the entire site has been impacted by the former industrial operations.

Site soils are contaminated with volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), ammonia, cyanide, and metals at concentrations exceeding the MDEQ industrial criteria for indoor and ambient air, direct contact, particulate inhalation, and surface water protection. Site groundwater is contaminated with VOCs, SVOCs, ammonia, cyanide, and metals at concentrations exceeding the MDEQ industrial criteria for indoor air, direct contact, and surface water protection.

Honeywell, the current owner of the Detroit Coke Site and the primary responsible party, has installed a demarcation membrane in certain areas, and approximately 15 to

30 cm (6 to 12 inches) of clean fill material has been placed over the membrane to prevent contact with the impacted soil. However, this membrane and clean fill layer may not be present throughout the entire site. Honeywell has also installed groundwater collection trenches to limit impacted groundwater from discharging to the Rouge River and Detroit River.

#### **14.6.1.2 Crossing X11(C)**

The former Revere Copper and Brass site occupies the southern portion of the X-11(C) Crossing between Jefferson Avenue and the Detroit River and was used for manufacturing copper and brass products from the early 1900's until 1985. In addition, significant portions of the site were filled with debris resulting from land reclamation on the site. Contamination generally consisting of VOCs, SVOCs, metals and polychlorinated biphenyls (PCBs) remain at the site in excess of Michigan Department of Environmental Quality (MDEQ) Part 201 Residential, Commercial and Industrial criteria.

### **14.6.2. Contaminated Sites in Canada**

There are numerous properties located along the shoreline of the Detroit River on the Canadian side which have a high potential to be contaminated sites.

#### **14.6.2.1. Crossing X10(A)**

The location of potential contaminated sites along Crossing X10(A) include the Nemark Plant (classified as a large industrial facility, former auto junkyard), the Brighton Beach Industrial Park and the Brighton Beach Power Plant.

#### **14.6.2.2. Crossing X10(B)**

The location of potential contamination sites for Crossing X10(B) includes those identified under Crossing X10(A), the Keith Transformer Station and West Windsor Power Plant. The Keith Transformer Station is also identified as a closed landfill.

#### **14.6.2.3. Crossing X11(C)**

The location of potential contamination sites for Crossing X11(C) includes the sites identified for Crossings X10(A) and X10(B) along with Vandehogen and Sterling Fuels, both of which are classified as former landfills. In addition, Sterling Fuels contains multiple fuel tanks and fuel lines. Crossing X11(C) could potentially impact the Lou Romano Reclamation Plant.

## **14.7. Foundation Compatibility with Existing Soils**

The very heavy foundation loads of the main river crossing require deep foundations to carry these loads into bedrock. Bedrock is expected at a depth of around 35 m (115 feet). Drilled shafts are expected to provide the most efficient way to carry the vertical foundation loads. The silty clays of the overburden are anticipated to be able to resist lateral loads under the main towers. Sunken caissons or braced excavations acting as shear walls may be required to resist the large lateral loads at the cable anchorages of suspension bridge options. Driven pile foundations are not expected to be economical at the main pier or cable anchorages due to the

large number of driven piles required. Also, damage to the piles during driving into and through hard pan may not make this foundation type a feasible solution.

Measures to control hydrogen sulfide gas, methane gas and soil contamination are required for all foundation types where soil is being excavated (drilled shafts, sunken caissons and braced excavations). At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 being most compatible.

## **14.8. Technical Challenges**

The X11(C) and X10(B) crossings involve spans of moderate length for both the cable-stayed and suspension alternates, with the exception of Option 4 which is a major cable-stayed bridge. These crossings do not pose unprecedented technical challenges based on an assessment of scale, location, geology or site. At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 being least challenging.

However, the X10(A) crossing with its skewed alignment and 925 m minimum span length, though not out of the ordinary for a suspension bridge would require a cable-stayed span that would be 35 m (115 ft) longer than the current world record span of the Tatar Bridge (890 m, 2920 ft), although shorter than two cable-stayed bridges currently under construction in China. Even though longer spans are under construction, technical challenges encountered during the design would be more significant than that of the moderate span suspension bridge, in addition to the construction risk, cost premiums, and schedule impacts that are discussed elsewhere in this document.

## **15. Safety and Security**

### **15.1. Risk to Structure**

For purposes of assessing the vulnerability of the proposed options to security or other related incidents at adjacent industrial facilities, the following properties have been identified as warranting further study: the Mistersky Power generating facility, the LaFarge Concrete facility, Sterling fuels transfer station, the Brighton Beach Power Generating Facility, and the Keith Transformer Station. At the Type Study phase this will be evaluated by counting the number of major industrial sites occurring adjacent to each crossing location, in both the U.S. and Canada, and then evaluating the relative risk these present to the safety and security of the crossing using professional judgment on a scale of 1 to 5. At the Conceptual Design phase a more rigorous risk analysis will be performed. Transport Canada and the RCMP are conducting a risk assessment related to Canadian side properties.

#### **15.1.1. Mistersky Power**

Immediately to the north of the X11(C) crossing lies the Mistersky Power Station, an oil-fired power generating facility owned by the city of Detroit and operated by the Department of Public Lighting. With the abundance of highly flammable material present on site, the facility may pose a potential hazard to the crossing structure if a significant event were to occur. This scenario is recommended to be studied further.

### **15.1.2. LaFarge Concrete**

Approximately 100 m north of the U.S. side of the X10(B) crossing is the LaFarge North America cement distribution center. The predominant feature of the site and the closest structure to the crossing is North America's largest cement silo (180 ft high and 95 ft in diameter). The presence of large amounts of fine particles in the silo represents an explosion hazard. In addition the facility is serviced by rail which would travel beneath the proposed structure. Finally, commercial ships dock at the facility which would impact the feasibility of a U.S. river pier and present a potential hazard to the bridge. This should be studied further.

### **15.1.3. Sterling Fuels**

To the north of the Canadian side of X11(C) lies the Sterling Fuels ship refueling depot. This site lies directly under the crossing.

### **15.1.4. Brighton Beach Power Generating Facility**

Brighton Beach Power, Inc. operates a gas-fired power plant between the alignments of the X10(A) and X10(B) crossings on the Canadian side of the river.

## **15.2. Risk to Residents**

The presence of a significant transportation link may present a risk to public safety. This risk may be from two sources; 1) accidental release of hazardous materials due to an accident; and, 2) from man-made incidences due to the possible targeting of a major piece of infrastructure. The risk will be proportional to the probability of an incidence and the population potentially exposed. In the Type Study phase of the project we will use as a surrogate the number of dwelling units within 0.8 km (½ mile), which is suggested by FHWA for the consideration of hazardous materials, to evaluate this criterion.

In the Conceptual Design phase we will use the criteria established by the Federal Highway Administration for the Routing of Hazardous materials. On the Canadian side the RCMP and Transport Canada are conducting a threat assessment for the Canadian side properties.

## **15.3. Emergency Response**

There are two groups at risk due to natural or man-made events on the bridge structure. Those on the facility, the traveling public, and those living in close proximity to the facility, as discussed in Section 15.2. A means of mitigating the risks associated with these events is prompt emergency response. On a structure of this nature it is common practice for emergency responders to have mutual assistance agreements where for instance fire fighters will fight a vehicle fire from the uphill side of the bridge. Emergency response time will be evaluated by determining the travel distance from the nearest public safety facility to the center of the bridge from both the U.S. and Canada.

## **15.4. Navigation Radar Impacts**

As discussed in Section 3.2 the Detroit River has a significant amount of commercial marine traffic. At night and in inclement weather those vessels rely on marine radar to safely transit the river.

A structure across the Detroit River will impact the navigation radar of the commercial vessels transiting the river. A major structure presents an impediment, or wall, to the radar creating a dead zone or clutter. In the case where a structure is perpendicular, or nearly so, to the channel this dead zone is relatively narrow. However, as structures become more skewed to the channel this dead zone becomes larger. For the Type Study the dead zone will be measured in meters along the channel center line from the point where the structure crosses the shoreline in the U.S. to the point where the structure crosses the shoreline in Canada.

## **15.5. Vulnerability/Redundancy**

At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 representing the least vulnerability.

### **15.5.1. Man-made**

The proposed Detroit River International Crossing Bridge is being planned and will be constructed in a post 9/11 world, and as such will need to consider the possibility of and mitigation for intentional acts to disrupt or destroy the structure. Clearly, a detailed threat, risk and vulnerability analysis of the proposed crossing is beyond the scope of this study, however, certain general vulnerability/redundancy risks should be considered where they may influence the choice of bridge type.

In general any of the structure options that are being considered can be designed to meet requirements for redundancy, strength, and toughness as may be prescribed as part of a detailed threat and vulnerability analysis. Fundamentally, non-redundant or fracture critical members should not be used. As much offset distance from the traveling public to critical bridge members should be provided as possible. Access should be limited and specific security monitoring measures considered.

Future analysis should consider the redundancy and appropriateness of the recommended structure types related to safety and security. Potential types of threats should be considered and reviewed with regard to each type of structures ability to address the threat.

Other unintentional "man-made" incidents such as accidents and fire may also present a risk to the structure.

#### **15.5.1.1. Fire**

The design will include this as a design condition and any of the bridge options will be appropriately designed for this condition. There would be no difference between options. For risk of fire beneath the structure, the options that cross a potentially hazardous site present a higher risk, unless that risk is mitigated. For example, on the Canadian shore, alignment X10(B) crosses the Sterling Fuels Depot that has pipelines carrying volatile fuels. It is considered that any risk from these crossings will be separately mitigated, either by restricting the presence of such materials below the bridge, by capping the fuel lines or by analysis to show that the risk is acceptable and therefore it is not considered that fire will be a significant factor in bridge type selection.

### **15.5.1.2. Vehicular Impact**

Risk to vehicular impacts is considered similar to the various bridge options and not a factor on the type selection process.

It is not considered that these considerations will be a differentiator between bridge type options, rather, they will be design criteria applied to all bridge options.

### **15.5.2. Natural**

The crossing may be subjected to a variety of natural extreme event hazards that include threats from earthquake, extreme winds, vehicular impacts, vessel impact (for options that have a tower in the river), and fire. In general, it is not considered that the effects of the design for natural hazards should be a significant factor in the choice between the bridge options. These are discussed as follows:

#### **15.5.2.1. Earthquake**

Either a suspension bridge or a cable-stayed bridge of the range of span lengths considered for the crossing options can be designed for the forced and displacements to respond to a seismic event. There is no differentiating risk for any of the structure options.

#### **15.5.2.2. Extreme Wind**

Long span bridges are susceptible to potential aeroelastic response to winds. These responses include vortex shedding, which does not represent a safety risk to the structure but may contribute to user discomfort or fatigue issues, buffeting, which can be a strength factor in the design, and flutter, which can lead to catastrophic failure of the structure under high winds. Any of the options considered will be subjected to exhaustive wind tunnel testing to assist with the design of the bridge and guard against all of these possible effects. In general, it is expected that the long span cable-stayed options may require more attention to the wind response and may present a more difficult engineering challenge, however it is not considered that response to extreme winds should be a significant factor in choosing the bridge option.

## **15.6. Vulnerability to Ship Collision**

The bridge options presented in this study fall into one of two categories: either they clear span the Detroit River and have no piers in the waterway, or they have one of the main bridge piers (towers or pylons) in the waterway. In the former case, there is no vulnerability to vessel collision and this section does not apply. In the later case, the pier in the waterway is to be designed in conformance with the AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges, specifically, either the pier is designed to resist the computed impact load, or a separate structure is provided to resist the impact load and protect the pier. This separate structure may take the form of large diameter dolphins, a pile supported protective ring, or an artificial island. Given the significant vessel collision loads for this project, a pier protection system is most likely.

The cost of the pier protection is included as part of the Comparison Construction Cost Estimate. The purpose of this section is to address the safety and security aspect of vessel collision design, i.e., the risk of having a pier in the waterway.

In general, one can not (reasonably) design a bridge with a pier in the water such that the risk is the same as a clear span. The code defines structures as either "regular" or "critical" for purposes of defining the return period for the probability based computation of vessel collision loads. For a "regular" bridge the acceptance criteria for probability of an occurrence that would lead to collapse of the structure is a return period of 1,000 years. For a "critical structure" this return period is 10,000 years. This structure is classified as a "critical" structure due to its size, importance and value. While these may seem like unreasonably low risk numbers, the code cautions that for rare events, such as ship collisions, very large levels of uncertainty exist and estimated risk can not be equated with actual risk because probability and consequence estimates that make up the risk analysis may be necessarily inexact.

The following table summarized the bridge options that have towers in the waterway and any special conditions that are appropriate to each case.

*Table 13. Summary of Piers in Waterway*

Alignment	Type Study Option/ Sub-Option		Pier in Waterway	Comment
X10(A)	1	Option 1a	No	
	2	Option 2a	Yes	CAN side
	3	Option 4a	Yes	CAN side
X10(B)	4	Option 1a	No	
	5	Option 2a	Yes	CAN side
	6	Option 4a	No	
	7	Option 5a	No	
	8	Option 6a	Yes	CAN side
X11(C)	9	Option 1a	No	
	10	Option 2a	No	
	11	Option 3a	No	

At the Type Study phase this will be evaluated using professional judgment on a scale of 1 to 5, with 5 representing the least vulnerability.

## 16. Summary and Conclusions

### 16.1. Evaluation Methodology and Criteria

This section presents a summary of the Practical Alternative evaluation process and screening criteria for the Detroit River crossing bridge, which is detailed further in the Evaluation of

Practical Bridge Options Technical Memo [6]. The evaluation process will consist of two phases; Phase 1 is the structural Type Study (TS Phase); and, Phase 2 is the Conceptual Design (CD Phase). Other project components, the plaza, connecting roadways, and interchanges will be evaluated separately. The goal of the process is to identify a preferred bridge option as part of the Preferred Alternative. As such the highest rated bridge may not be the preferred bridge option as the evaluation of other project components will factor into the selection of a Preferred Alternative. However, the evaluation should yield a preferred bridge option for each crossing alignment.

The evaluation process will consist of scoring of screening criteria by competent bridge professionals from Parsons and URS with incorporation of Partnership input at appropriate times. At the conclusion of each development phase the Consultant Team will score each of the bridge options using the screening criteria discussed below. This will result in a ranking, based on the scores and professional judgment regarding the relative weight of each criterion that will be used for narrowing down the number of reasonable alternatives. At appropriate stages of the process other agencies, such as the US Coast Guard, Canada Coast Guard, Michigan State Historic Preservation Officer, etc. will be consulted for their input.

Because this process is iterative, i.e., the structures are at varying degrees of engineering development throughout the process, some screening criteria may not be applicable in TS Phase but will be applicable in CD Phase. Likewise, individual Performance Factors may not be applicable in the TS Phase. Also, in the CD Phase the methodology to develop the metrics for each of the screening criteria will become more detailed. For example in the TS Phase the Construction Cost will be based on the scaling of the design of similar structures. In the CD Phase, actual design of major structural elements will be performed and the cost will be based on quantity estimates. In addition, a quantitative statistical analysis of the costs given known risk factors will be performed.

Below is a summary of the screening criteria to be used to evaluate the alternatives in the Type Study and Conceptual Design Phases. Each Screening Criteria will be evaluated using several Performance Factors, described in **Section 16.2**.

*Table 14. Screening Criteria.*

Screening Criteria	Practical Alternative Phase	
	Type Study	Conceptual Design
Initial Cost	X	X
Life-Cycle Cost	n/a	X
Constructability	X	X
Aesthetics	n/a	X
Safety and Security	X	X

## **16.2. Evaluation Data**

A full matrix of all of the evaluation criteria can be found in **Appendix F: Evaluation Matrix**. Below is a summary of the evaluation data.

### 16.2.1. Initial Cost

Table 15. Construction Cost and Cost Risk Evaluation Data.

Crossing	Type Study Option	Bridge Type	River Pier	Construction Cost Estimate – 2006 US\$ (000,000's)	Cost Risk (Scale 1 -5)
X10(A)	Option 1	Susp.	N	770 - 920	2Error! Not a valid link.
	Option 2	Susp.	Y	680 - 810	4
	Option 3	CS	Y	620 - 740	1
X10(B)	Option 4	CS	N	430 - 510	2
	Option 5	CS	Y	370 - 440	3
	Option 6	Susp.	N	480 - 550	5
	Option 7	Susp.	N	470 - 540	5
	Option 8	Susp.	Y	420 - 490	4
X11(C)	Option 9	CS	N	450 - 530	3
	Option 10	Susp.	N	500 - 580	5
	Option 11	Susp.	N	520 - 600	5

### 16.2.2. Constructability

Table 16. Constructability Evaluation Data.

Type Study Option	Duration (months)	Risk (Scale 1-5)	Disruption (Scale 1 -5)		Major Utilities			Contamination Sites		Foundation Compatibility (Scale 1 -5)	Technical Challenges (Scale 1 -5)
			U.S.	Can.	# U.S.	# Can.	Risk (Scale 1-5)	#	Risk (Scale 1-5)		
<b>Crossing X10(A)</b>											
Option 1	62	2	3	4	3	3	4	2	4	4	3
Option 2	56	2	3	4	3	3	4	2	4	4	3
Option 3	55	2	3	4	3	3	4	2	4	4	2
<b>Crossing X10(B)</b>											
Option 4	51	4	2	3	3	3	4	2	4	4	2
Option 5	43	3	2	3	3	3	4	2	4	4	3
Option 6	52	4	2	3	3	3	4	2	4	4	3
Option 7	49	4	2	3	3	3	4	2	4	4	3
Option 8	43	3	2	3	3	3	4	2	4	4	3
<b>Crossing X11(C)</b>											
Option 9	47	5	4	2	1	1	4	5	4	4	3
Option 10	42	4	4	2	1	1	4	5	4	4	3
Option 11	51	4	4	2	1	1	4	5	4	4	3

### 16.2.3. Safety and Security

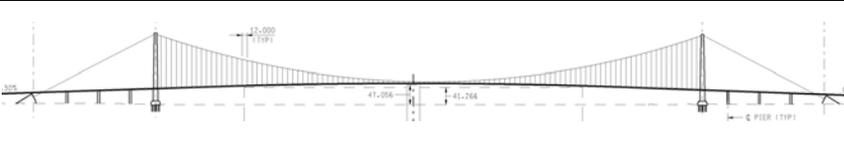
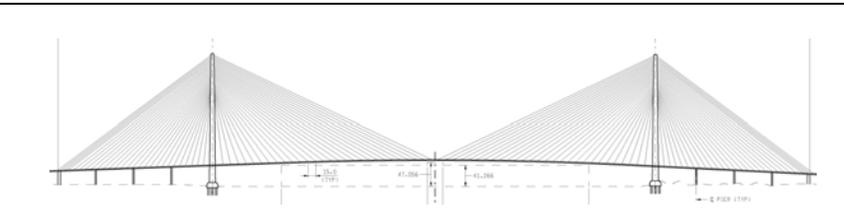
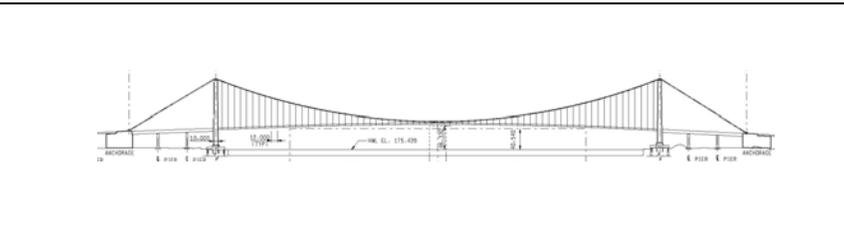
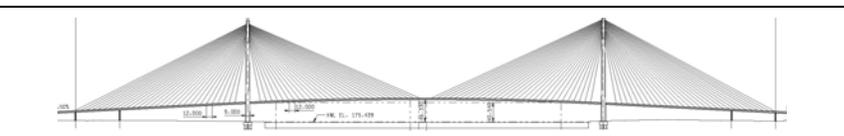
Table 17. Safety and Security Evaluation Data.

Type Study Option	Risk to Bridge (# Industries)			Risk to Residents		Emergency Response		Navigation Interference (m)	Vulnerability (Scale 1 -5)		
	# U.S.	# Can.	Risk (Scale 1-5)	U.S. (DU's)	Can. (DU's)	U.S. (km)	Can. (km)		Man-Made	Natural	Ship Impact
<b>Crossing X10(A)</b>											
Option 1	2	1	3	118	14	3.8	7.3	641	3	4	5
Option 2	2	1	3	118	14	3.8	7.3	641	3	4	3
Option 3	2	1	3	118	14	3.8	7.3	641	3	4	3
<b>Crossing X10(B)</b>											
Option 4	1	1	3	131	10	3.6	7.9	338	3	4	5
Option 5	1	1	3	131	10	3.8	7.9	338	3	4	3
Option 6	1	1	3	131	10	3.8	7.9	338	3	4	5
Option 7	1	1	3	131	10	3.8	7.9	338	3	4	5
Option 8	1	1	3	131	10	3.8	7.9	338	3	4	3
<b>Crossing X11(C)</b>											
Option 9	1	1	2	102	600	3.8	9.1	291	2	4	5
Option 10	1	1	2	102	600	3.8	9.1	291	2	4	5
Option 11	1	1	2	102	600	3.8	9.1	291	2	4	5

### 16.3. Summary

Cost, cost risk, schedule duration, schedule risk, and vulnerability to ship impact were considered to be the major differentiators between options at each crossing alignment after an evaluation of the data presented above. Some evaluation factors did not vary from option to option along an alignment. The following TS Options are recommended for further consideration and study.

*Table 18. Recommended retained Type Study Options.*

Type Study Option Elevation	Type Study Option
<b>X10(A)</b>	
	Option 1
<b>X10(B)</b>	
	Option 4
	Option 7
<b>X11(C)</b>	
	Option 9
	Option 10

**16.3.1. Options Retained for Study**

In order to maintain a consistent approach to the development and evaluation of bridge options throughout the Practical Alternative phase of the study it is recommended that two options be retained at each crossing alignment (except crossing X10(A) where there is only one viable option). While it is recommended, from a technical perspective, that these options be retained for further study, as discussed earlier, it is recognized that Crossing X10(A) is not preferred from a bridge engineering perspective. Therefore, consideration will be given to postponing the advancement of the conceptual design for crossing X10(A) until preliminary results are obtained from the geotechnical investigation program and any other relevant project EA/EIS studies.

The final recommended options, presented in this report and based on data received to date, clear-span the river and do not have piers in the water. Although options with piers in the water were on the order of \$60 to \$110 million less costly than equivalent structure

types without marine piers, input from both the U.S. and Canadian Lake Carriers Association, River Pilots, and the U.S. Coast Guard made strong objection to piers in the river citing navigation issues related to docking on both the U.S. and Canadian shores and navigation entering and exiting the River Rouge. Their objections were considered compelling and led to recommendation at all locations to clear span the river. Table 18 presents the final recommended options for each alignment.

This section will summarize the reasoning for the recommendations to retain the TS Options.

#### Crossing X10(A) – Type Study Option 1

The rationale for including an option from the X10(A) alignment is discussed above. The only option that meets the requirements of avoidance of piers in the water is the suspension bridge, Study Option 1.

#### Crossing X10(B) – Type Study Option 4 & 7

The alternatives selected for crossing X10(B) comply with the Coast Guard requirements to clear span the Detroit River. . For the suspension option it is more economical not to suspend the side spans in the case of Option 7 .

#### Crossing X11(C) – Type Study Option 9 & 10

Piers in the water at this crossing were not considered to be practical. Due to its economy it is recommended that the cable-stayed option be retained. For the suspension bridge type the elimination of suspended side spans saves on the order of \$20 million dollars while also improving the horizontal geometrics by reducing the length of tangent required. There may also be some safety gained by reducing the side span length in that the anchorage will be farther away from the petroleum storage tanks in Canada should they remain in service.

### **16.3.2. Options Dropped From Further Consideration**

This section will summarize the reasoning for the eliminating TS Options from further consideration at this time. It should be noted that although a particular option is dropped from consideration at this time changes in circumstances may warrant reviving an option. An example are the cable-stayed and suspension bridge options at Crossing X10(B) with both piers on land. At this location the options with piers in the water are not being carried forward for further study due to agency input. If at a future date the agencies reconsider this position and allow piers in the River, the bridge options at crossing X10(B) should be revisited.

#### Crossing X10(A) – Type Study Options 2 and 3

The requirement of avoiding piers in the Detroit River led to a recommendation to eliminate these options.

#### Crossing X10(B) – Type Study Option 5, 6 & 8

Options 5 and 6 have piers in the River which drew compelling navigation related objections from U.S. and Canadian Lake Carriers Association, River Pilots and the U.S. Coast Guard. Option 8 is similar to Option 7 except it has suspended side spans.

Our cost analysis indicates that the more economical design will be with the unsuspend side span arrangement.

Crossing X11(C) – Type Study Option 11

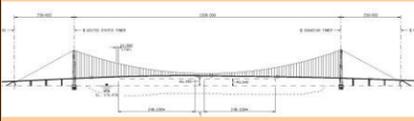
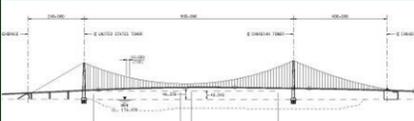
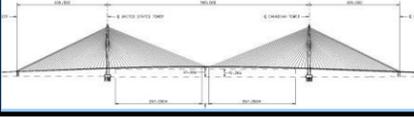
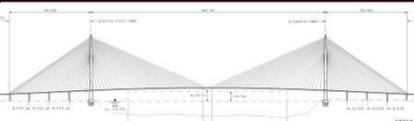
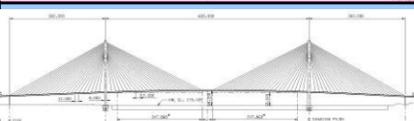
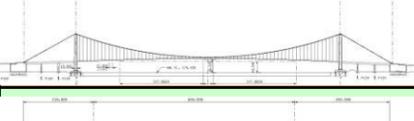
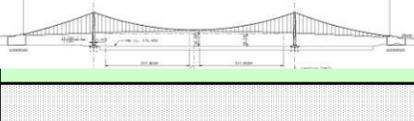
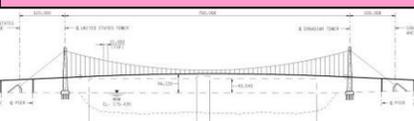
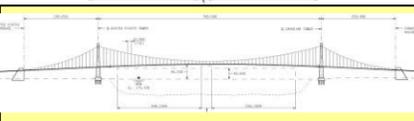
When the full suspension option is compared to the unsuspended side span option it does not have any performance benefits that would offset the additional cost.

## **Appendix A: Option Log, General Plan and Elevations**

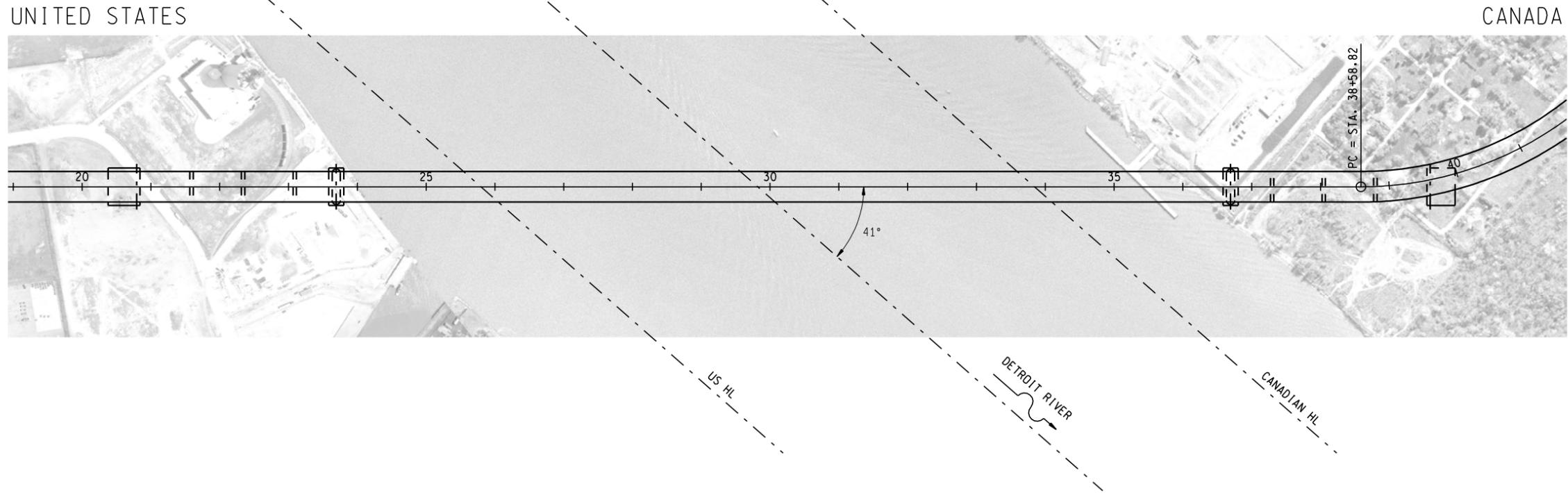
# Detroit River International Crossing Project

By: BLC  
Revised: 1/24/2007

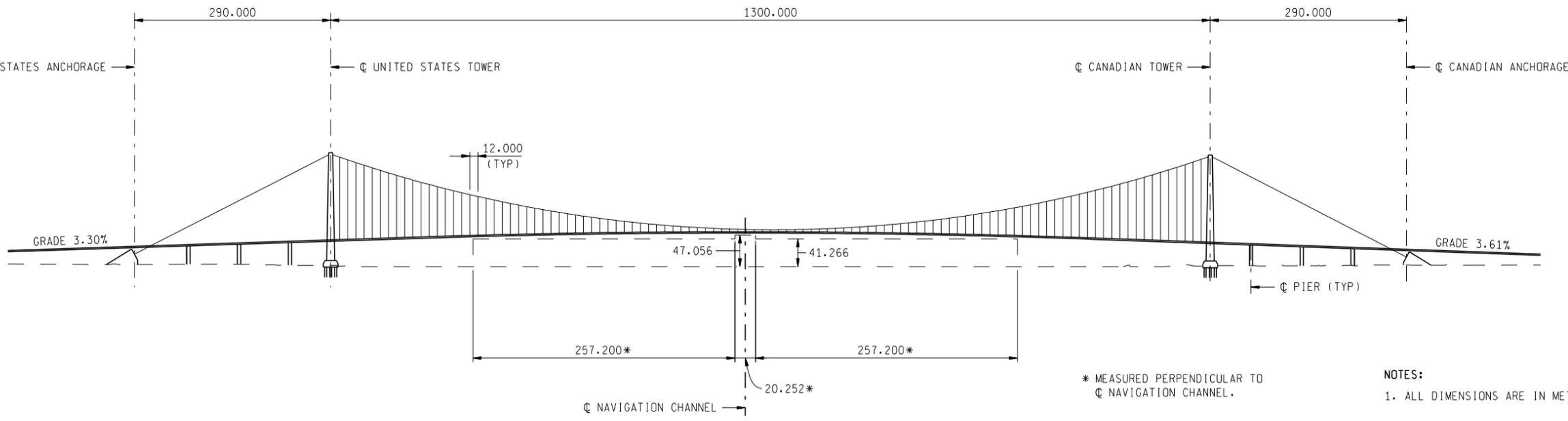
## Practical Alternatives - Bridge Type Option Log

TS Option	Thumbnail	Descriptive (Plan) Name	General Description	Spans			Assigned	Notes		
				US Anch	Ratio	Main				
<b>Alignment X10(A)</b>										
1		Crossing X10(A) Option 1a	Suspension; Land Piers, Portal-Pylon, Unsuspended Back Span	290	0.22	1300	0.223	290	URS	
2		Crossing X10(A) Option 2a	Suspension; CN River Pier, Portal-Pylon, Unsuspended Back Span	240	0.259	925	0.405	375	URS	
		Crossing X10(A) Option 3a	Suspension; US River Pier, Portal-Pylon, Unsuspended Back Span	370	0.37	1000	0.21	210	URS	<b>Dropped</b> due to proximity of river pier to River Rouge, Old Channel.
3		Crossing X10(A) Option 4a	Cable Stay; CN River Pier	416	0.45	925	0.45	416	URS	
<b>Alignment X10(B)</b>										
4		Crossing X10(B) Option 1a	Cable Stay; Land Piers, A-Pylon, Fan Arrangement Option 1	300	0.349	860	0.349	300	URS	
		Crossing X10(B) Option 1b	Cable Stay; Land Piers, A-Pylon, Fan Arrangement Option 2	300	0.345	870	0.345	300	URS	Combined with Option 1a
5		Crossing X10(B) Option 2a	Cable Stay; CN River Pier, Delta-Pylon, Fan Arrangement	282	0.47	600	0.47	282	Parsons CHI	
		Crossing X10(B) Option 3a	Cable Stay; US River Pier, Delta-Pylon, Fan Arrangement	307	0.472	650	0.472	307	Parsons CHI	<b>Dropped</b> due to proximity of river pier to River Rouge, Old Channel.
6		Crossing X10(B) Option 4a	Suspension; Land Piers, Portal-Pylon, Suspended Back Span	190	0.218	870	0.218	190	Parsons NY	
7		Crossing X10(B) Option 5a	Suspension; Land Piers, Portal-Pylon, Unsuspended Back Span	170	0.195	870	0.195	170	Parsons NY	
8		Crossing X10(B) Option 6a	Suspension; CN River Pier, Portal-Pylon, Suspended Back Span	210	0.35	600	0.47	282	Parsons NY	
		Crossing X10(B) Option 7a	Suspension; US River Pier, Portal-Pylon, Suspended Back Span	228	0.339	672	0.286	192	Parsons NY	<b>Dropped</b> due to proximity of river pier to River Rouge, Old Channel.
		Crossing X10(B) Option 8a	Suspension; CN River Pier, Portal-Pylon, Unsuspended Back Span	150	0.25	600	0.25	150	Parsons NY	Anchorage in river
<b>Alignment X11(C)</b>										
9		Crossing X11(C) Option 1a	Cable Stay; Land Piers, A-Pylon, Fan Arrangement option 1 & 2, viaduct approach	363	0.484	750	0.484	363	Parsons CHI	Alignment a1 longer tangent and long approach to CN Plaza B.
		Crossing X11(C) Option 1b	Cable Stay; Land Piers, Y-Pylon, Fan Arrangement option 1 & 2, viaduct approach	363	0.484	750	0.484	363	Parsons CHI	Combined with option 1a.
10		Crossing X11(C) Option 2a	Suspension: Land Piers, Portal Tower, Unsuspended Back Span	120	0.16	750	0.16	120	URS	Alignment a2 - long approach to CN Plaza B. Can also use alignment a4, which is shorter approach to CN Plaza C.
11		Crossing X11(C) Option 3a	Suspension: Land Piers, Portal Tower, Suspended Back Span	250	0.333	750	0.333	250	URS	Alignment a1 longer tangent and long approach to CN Plaza B. Can also use alignment a3, which is shorter approach to CN Plaza C.

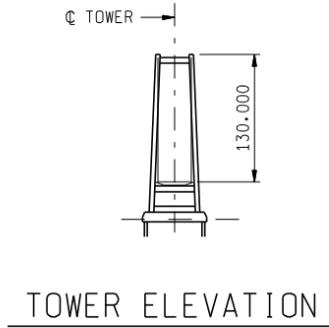
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\* MEASURED PERPENDICULAR TO Q NAVIGATION CHANNEL.

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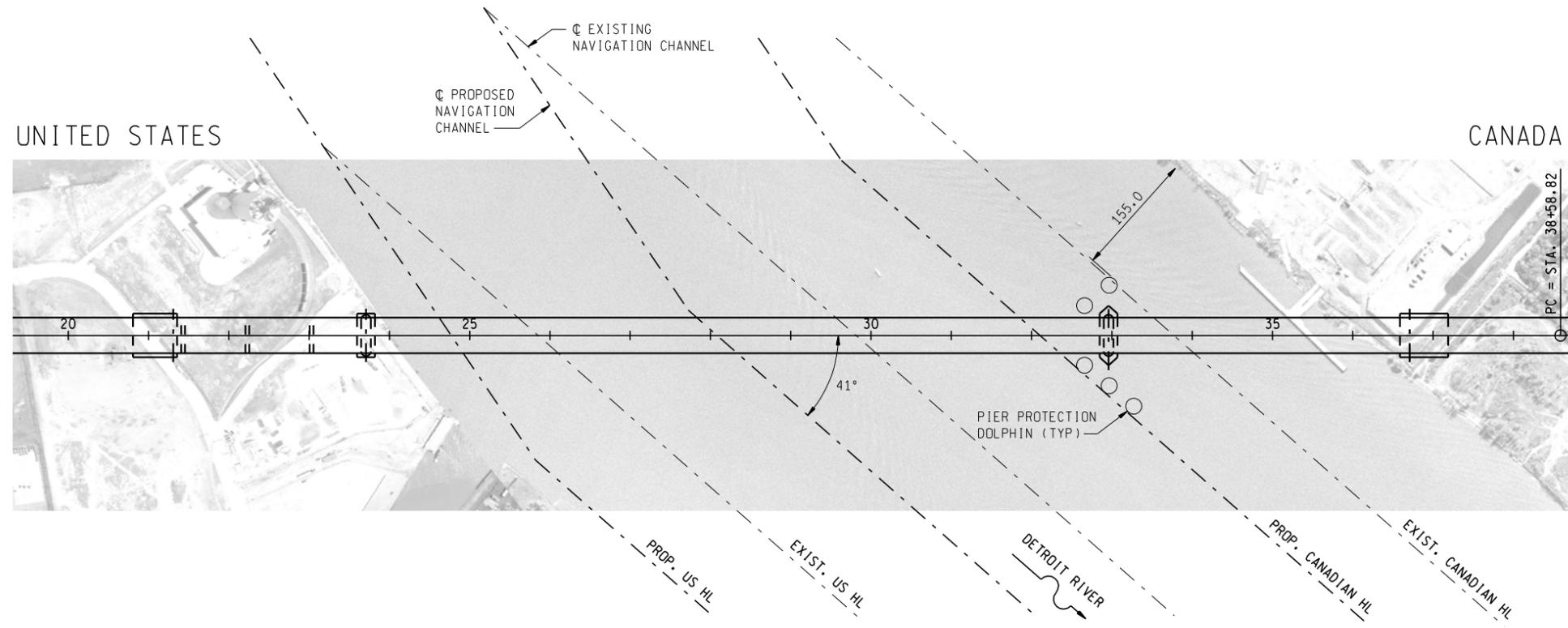


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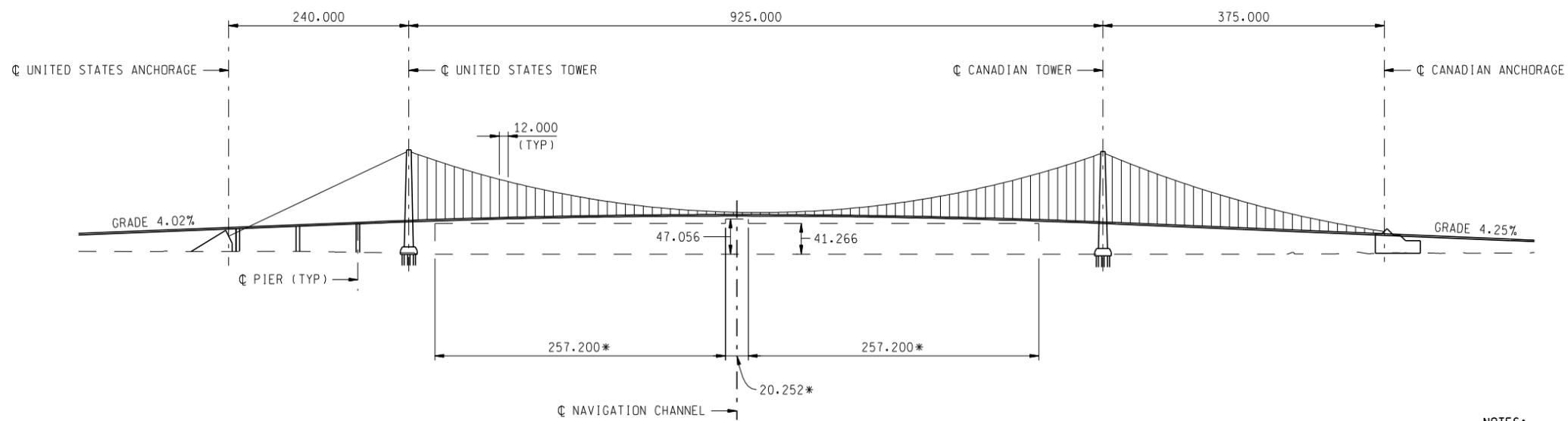
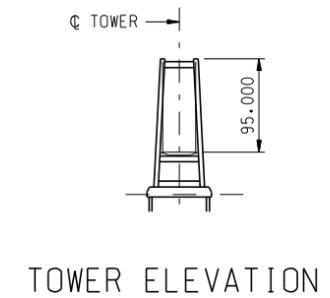
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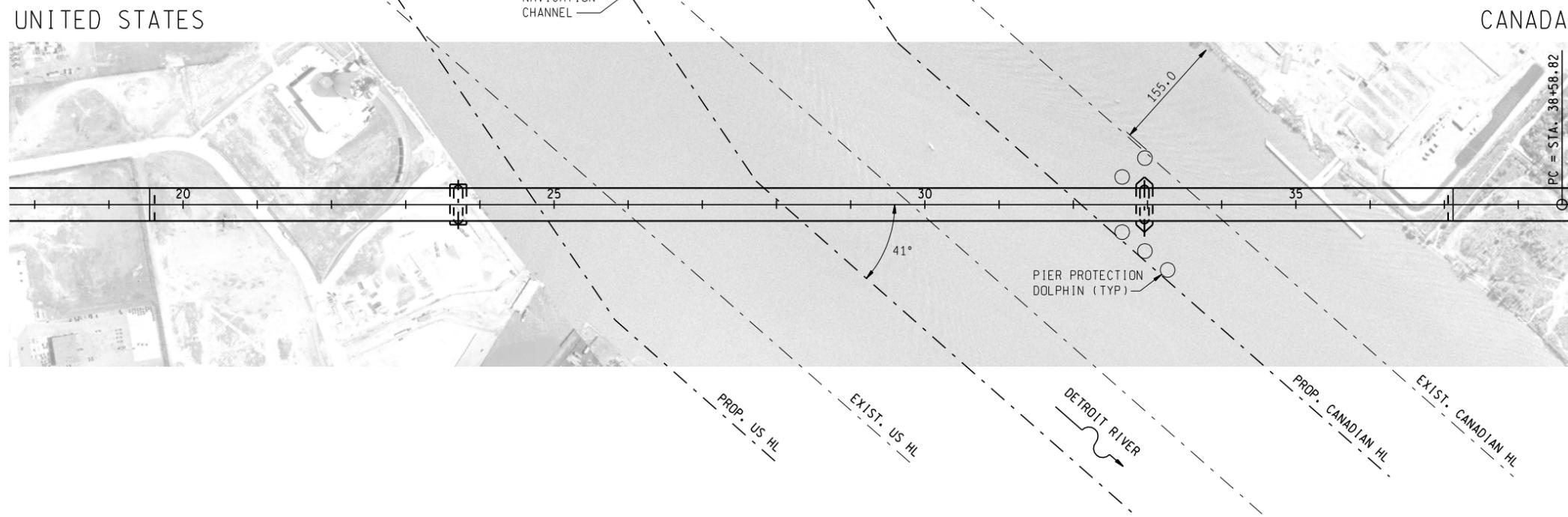
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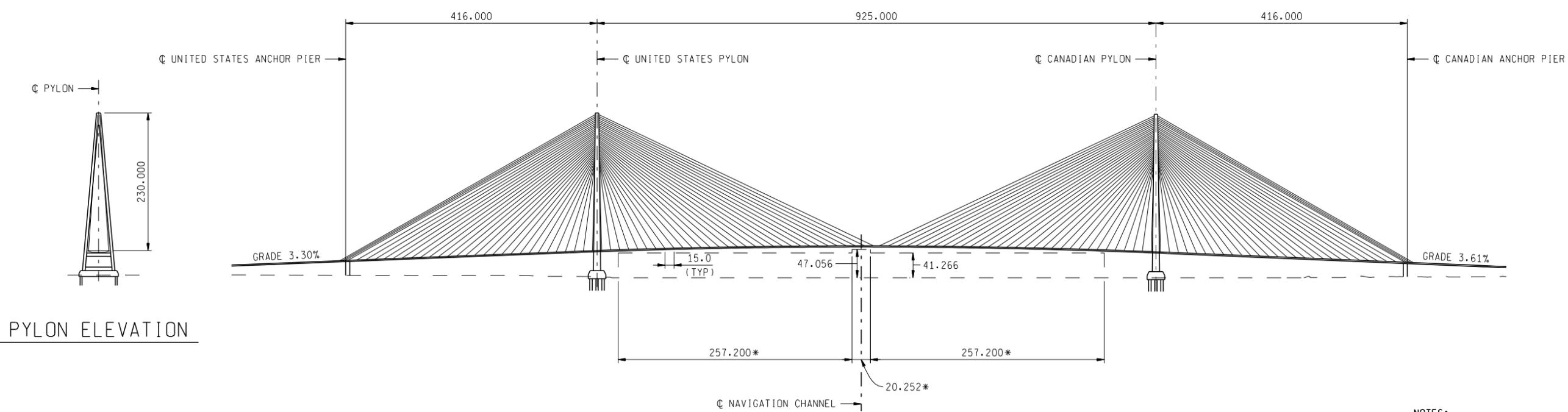
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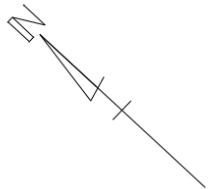


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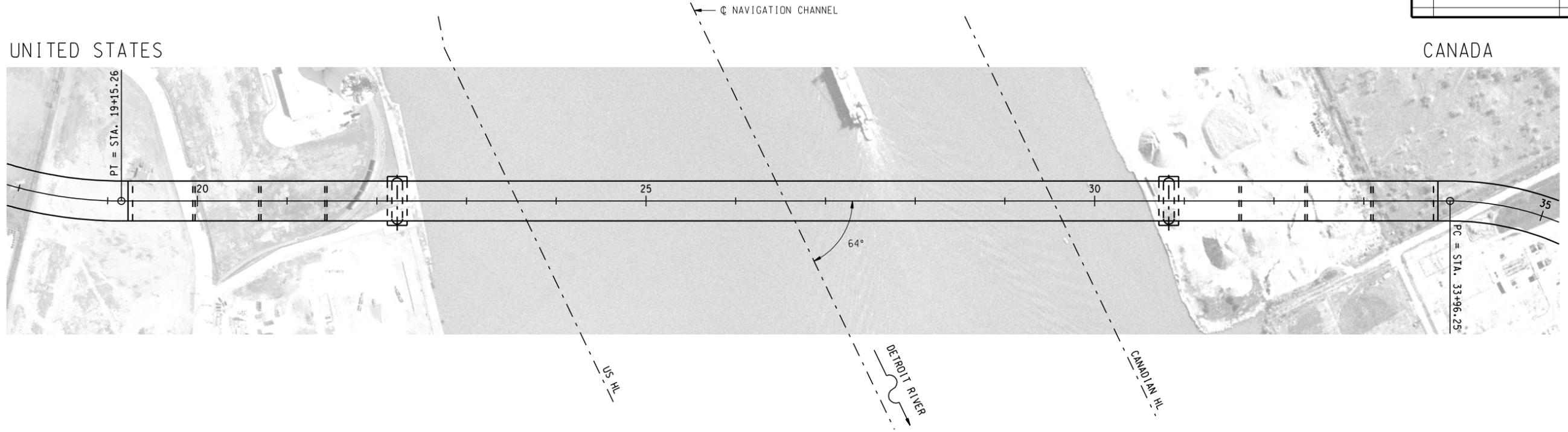
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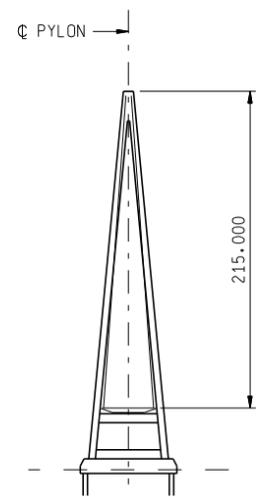


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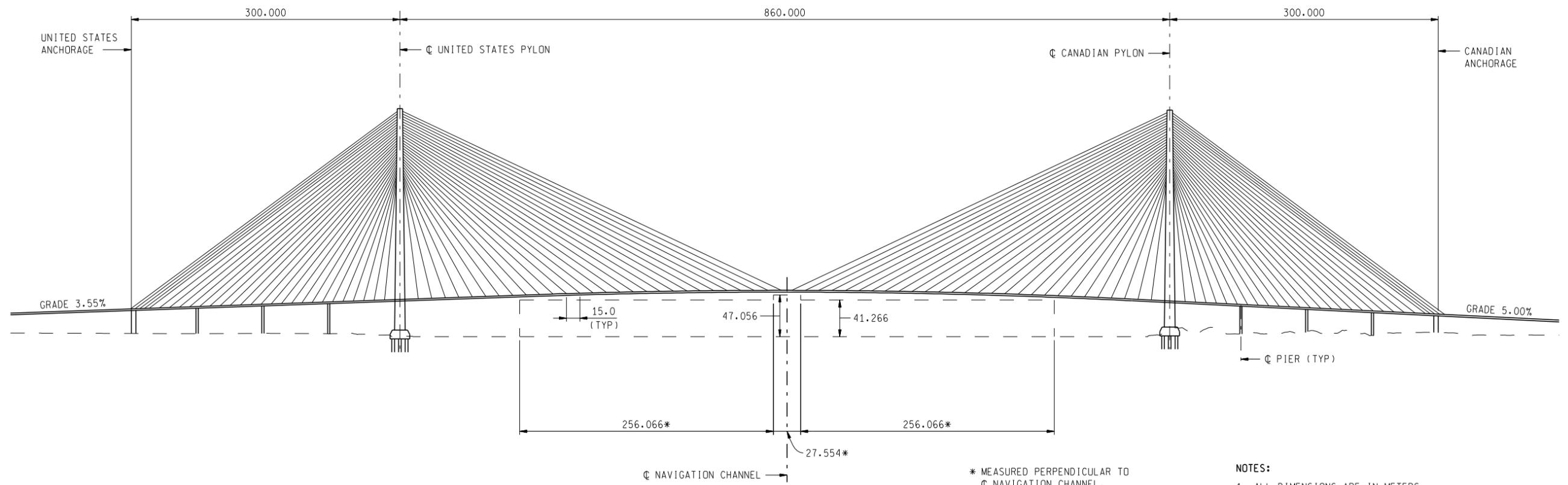
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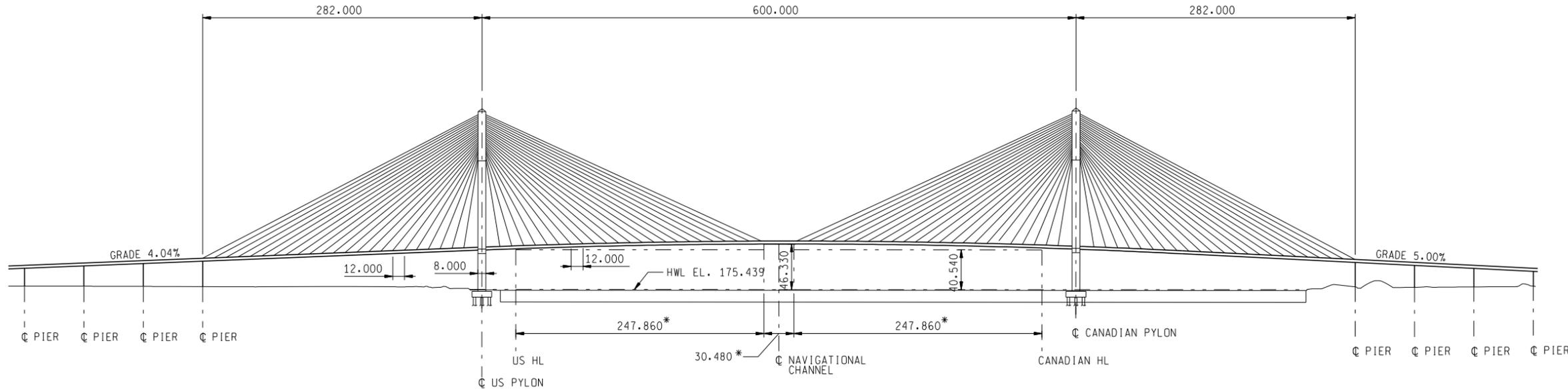
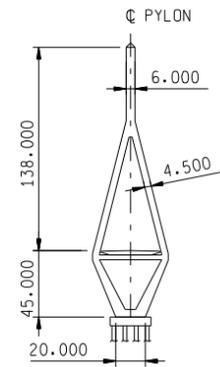
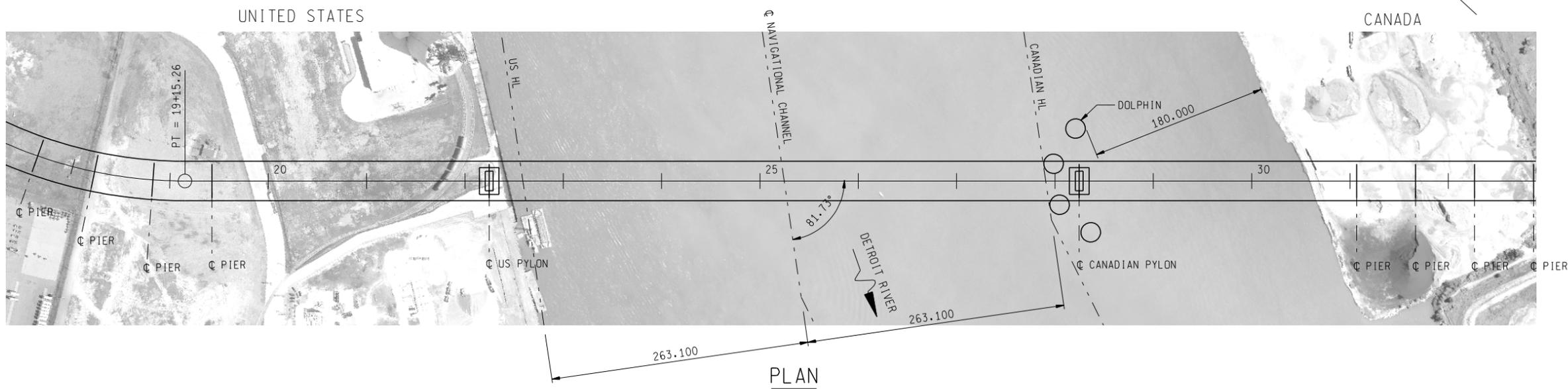
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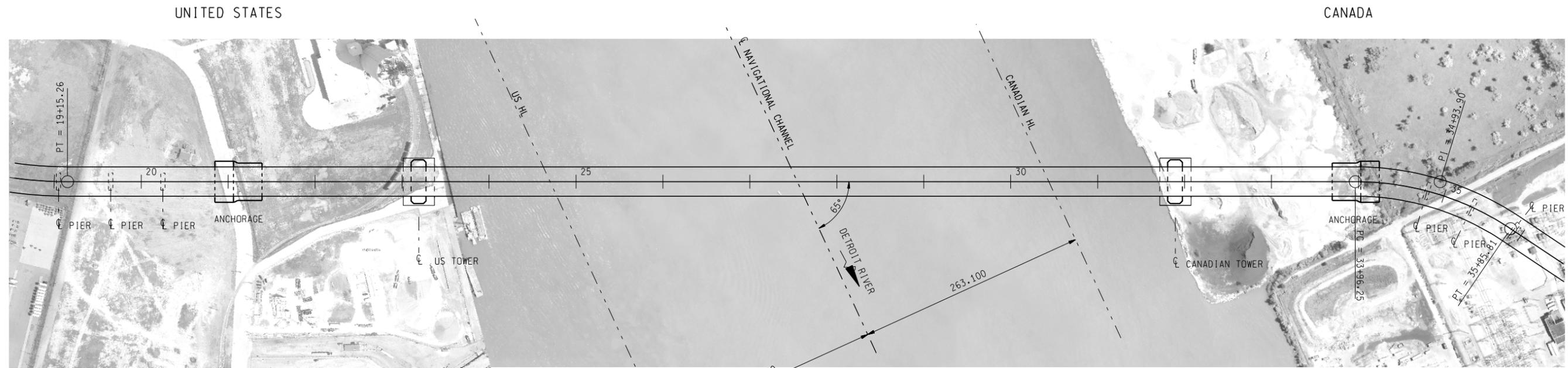
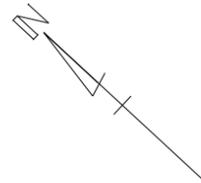
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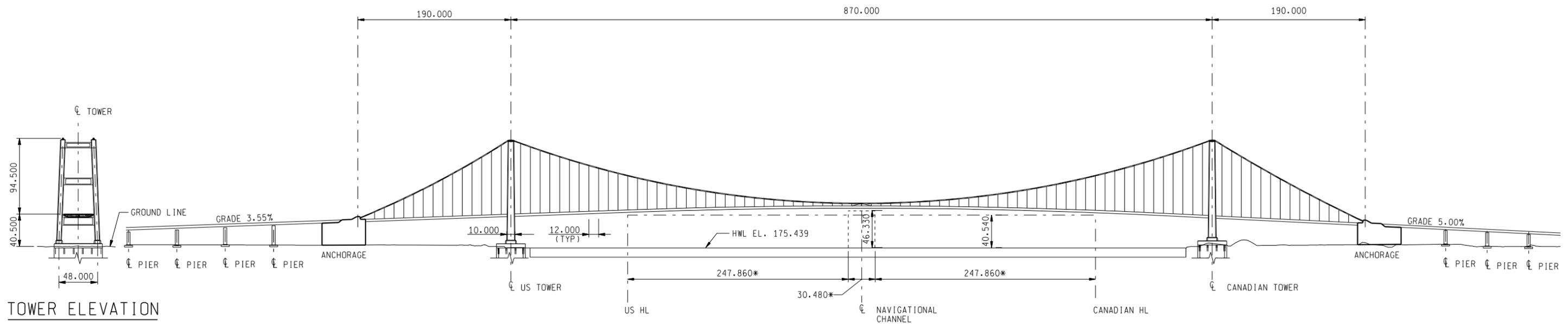
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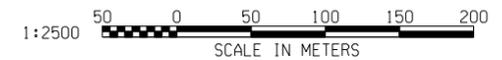


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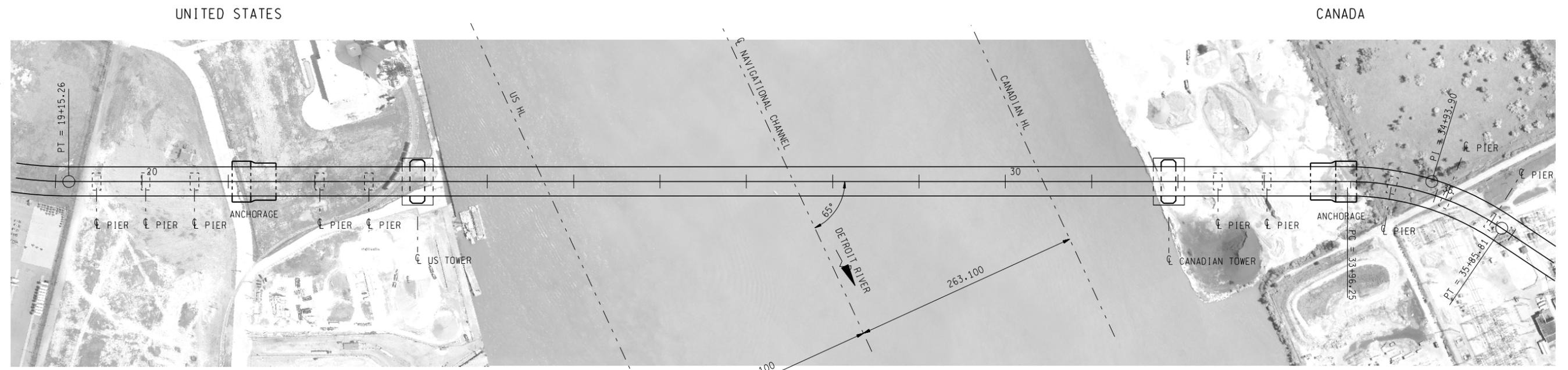
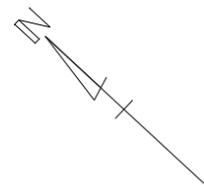


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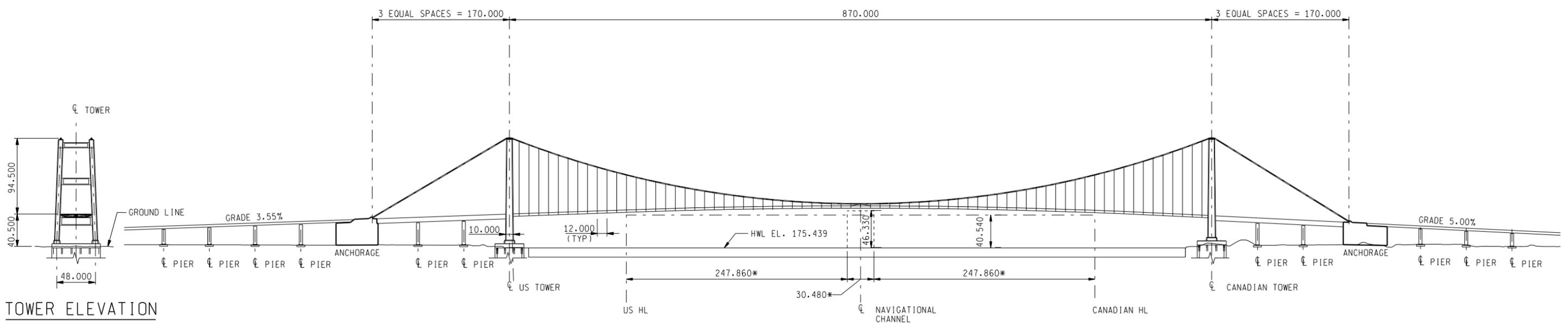
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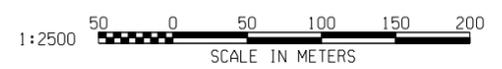
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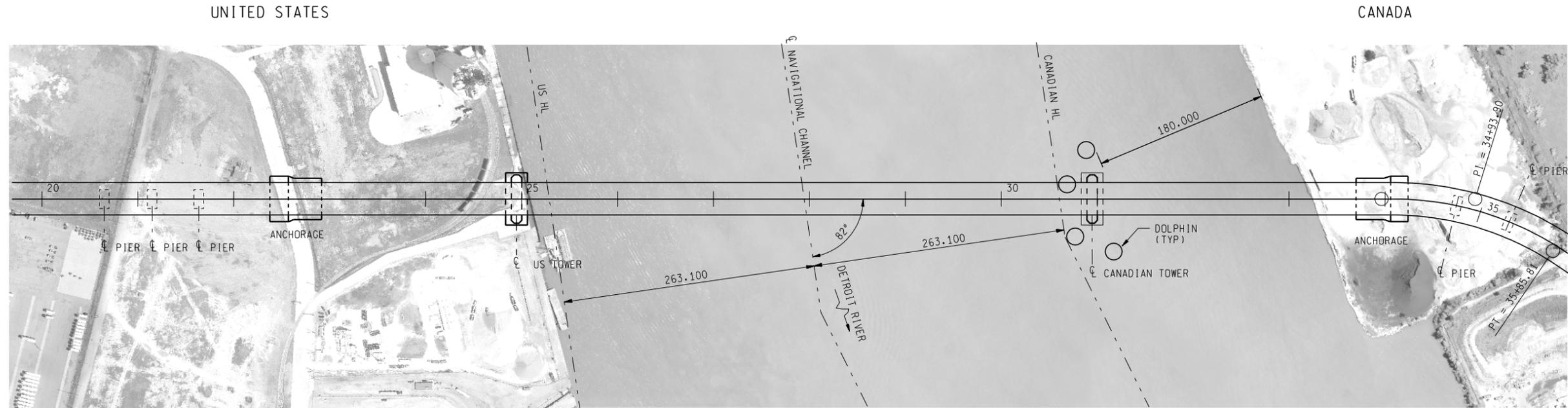
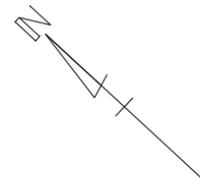


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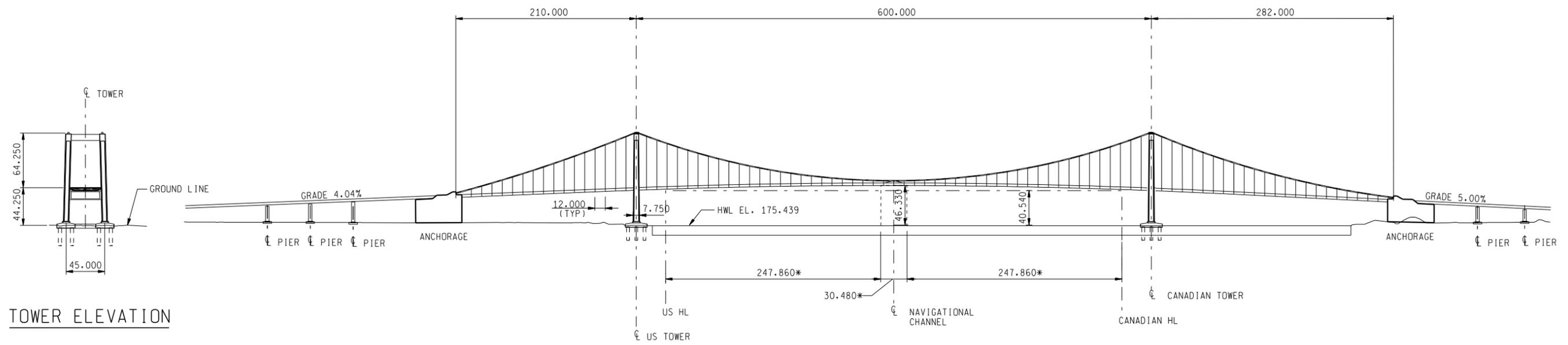
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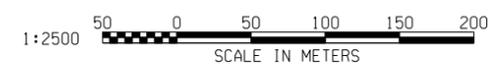


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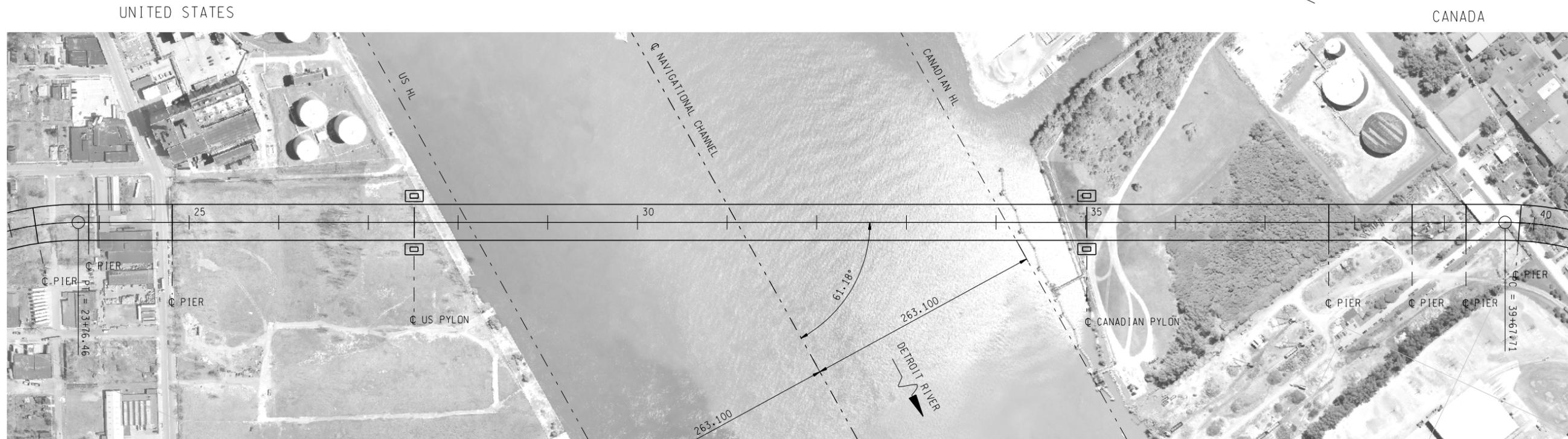


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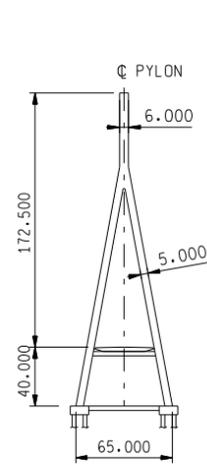
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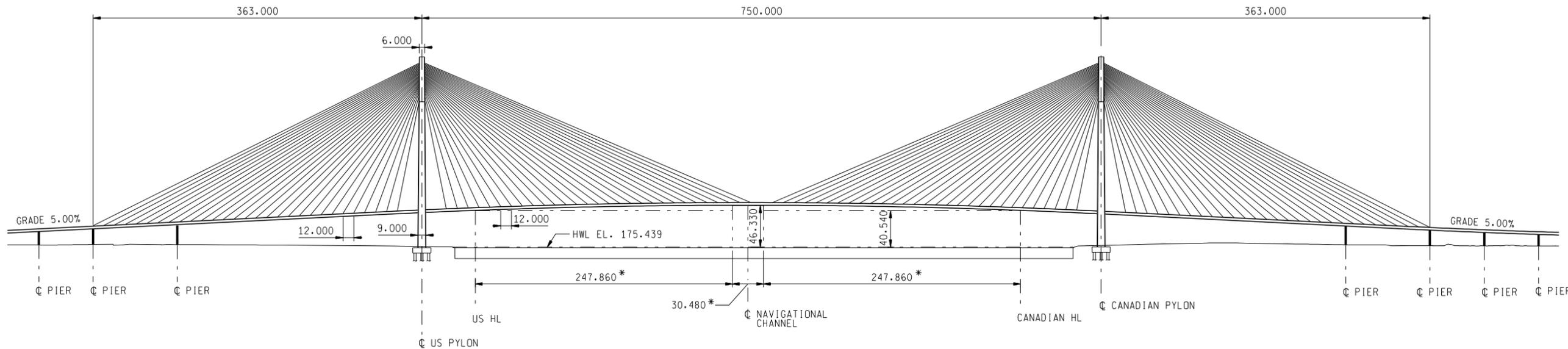
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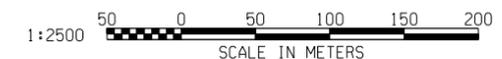
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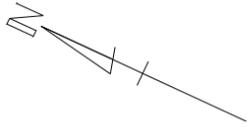
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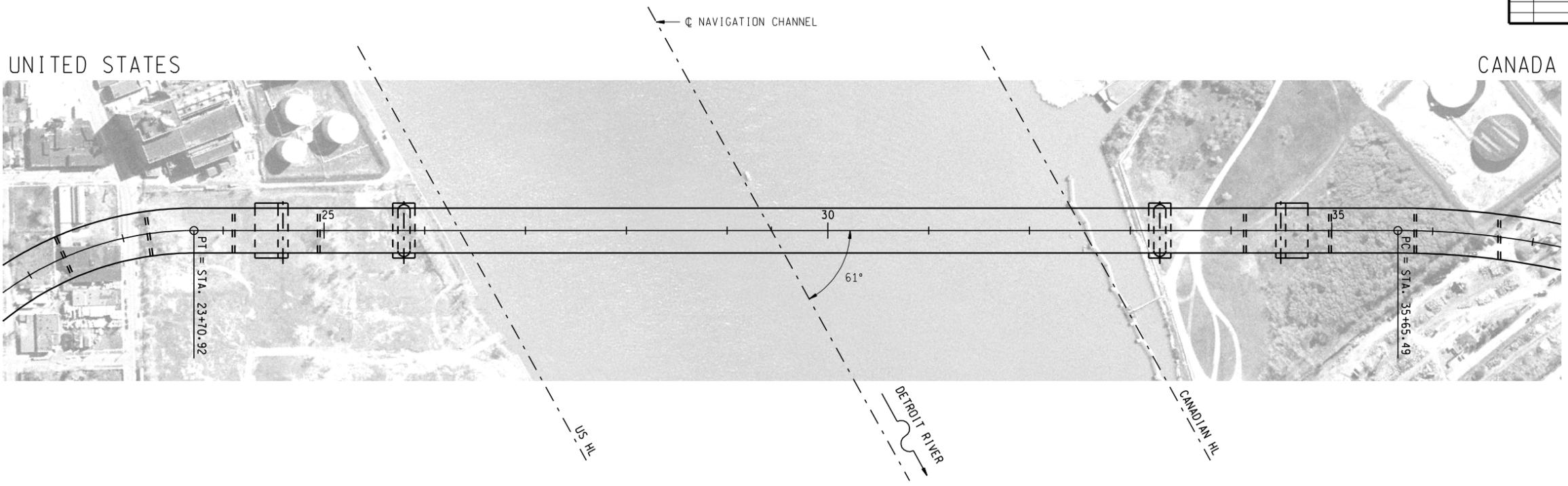
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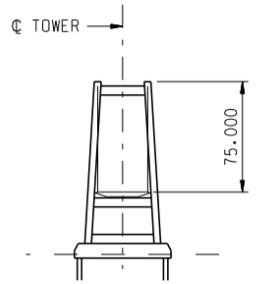


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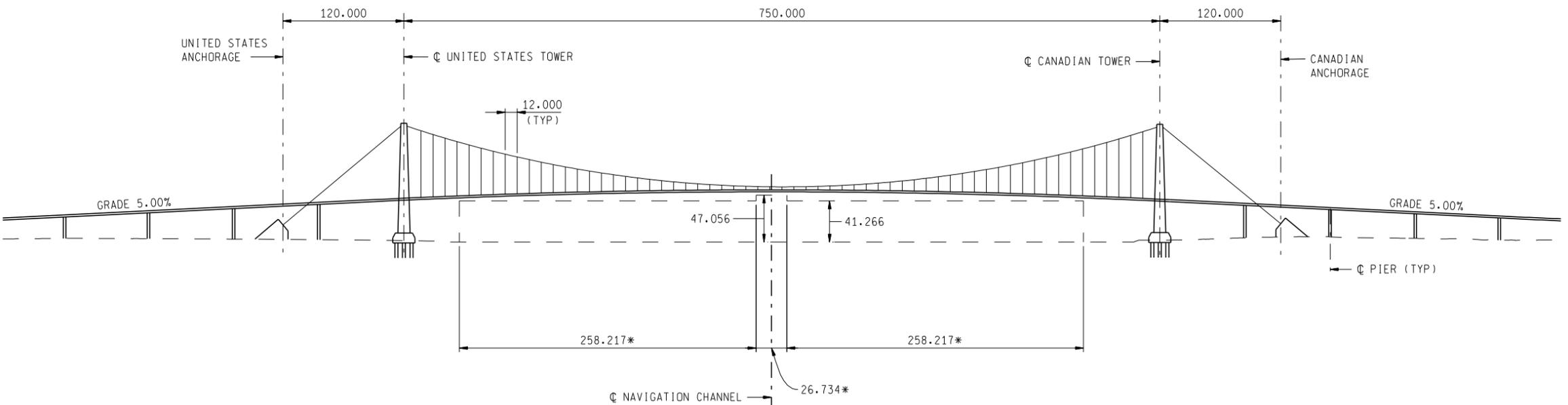
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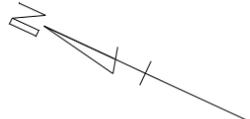


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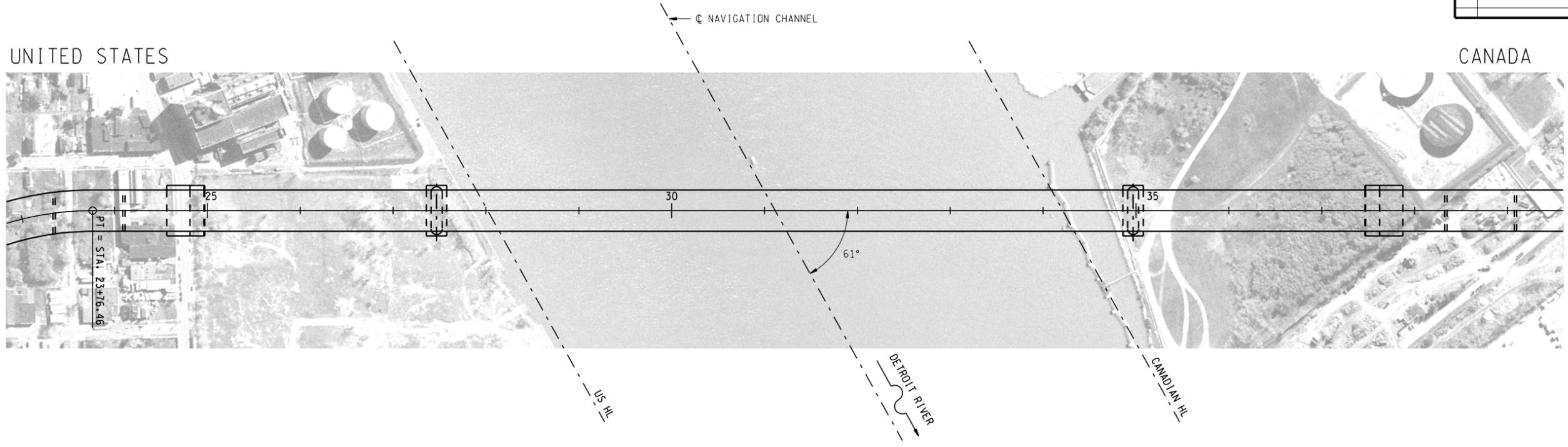
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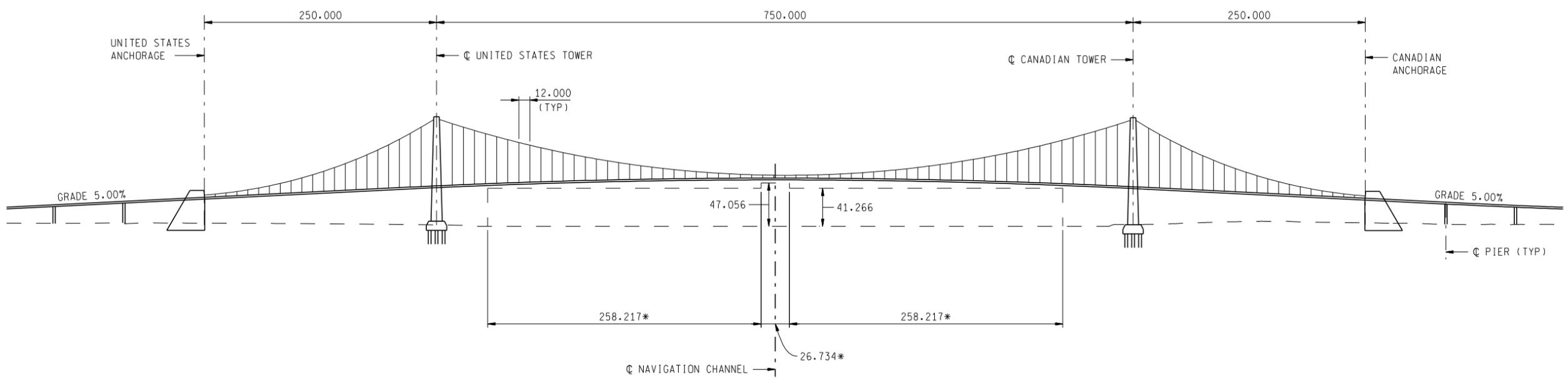


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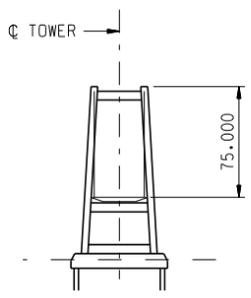
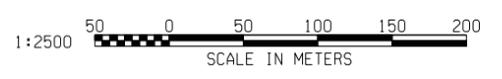
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## **Appendix B: Design Criteria**

Detroit River  
INTERNATIONAL CROSSING  
PROJECT

A BORDER TRANSPORTATION PARTNERSHIP



*Draft #3.1*

# Preliminary Type Study Design Criteria

For  
Detroit River International Crossing

September 7, 2006

Authored By:

**PARSONS**

In Cooperation With:

**URS**

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## 1. GENERAL

### 1.1. Objectives and Scope

This memo presents the general design criteria for the development of the main river bridge options in the Type Study phase of the Detroit River International Crossing Study. The criteria will be developed further for subsequent project phases. The project will be developed using the International System of Units (SI) units. The design of the main river crossing will be coordinated between Parsons and URS. 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.

### 1.2. Design References and Governing Criteria

The design shall meet the requirements of both the AASHTO *LRFD Bridge Design Specifications*, SI Units, 3<sup>rd</sup> Edition and the Canadian *Highway Bridge Design Code*, CAN/CSA S6-00 (S6), and in general the more restrictive code shall govern. Interpretations and exceptions are specified herein.

The following documents are used in the development of the Detroit River International Crossing Preliminary Type Study:

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

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

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

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

AASHTO *LRFD Bridge Design Specifications*, SI Units, 3<sup>rd</sup> Edition and all Interim Revisions

Canadian *Highway Bridge Design Code*, CAN/CSA S6-00  
*Geometric Design Standards for Ontario* (GDSOH)

PTI *Recommendations for Stay Cable Design, Testing and Installation*, 4th Edition, 2001

During conceptual design, additional specifications will be introduced as needed.

## 2. GEOMETRY AND CLEARANCES

### 1.2. General

Bridge options for the main river bridge will be developed and evaluated using

current MDOT, FHWA, and AASHTO geometric guidelines, policies, and standards for bridges as listed in Table 1. Where Ministry of Transportation Ontario (MTO) standards are greater than the U.S. standard the higher standard will be used. The geometric design criteria recommended for the river bridge reflects the assumption that it will function as a connection between the US and Canadian Plazas, both of which are secure facilities, with traffic entrances and exits to functional areas very close to the ends of the bridge. The design speed is 60 km/h.

## 2.2. Horizontal and Vertical Geometry

*Table 1*

Item	Criteria	References
Roadway Classification	Urban Principal Arterial	AASHTO Chapter 1 (p. 10-11)
Design Level of Service	LOS C LOS D minimum	AASHTO Exhibit 2-32 (p. 85) MDOT Practice
Design Speed (km/h)	60 km/h	AASHTO Chapter 2 (p. 67-72)
ADT for Year of Completion 2013	Not yet available	Traffic Report
ADT for Design Year 2035	Not yet available	Traffic Report
<b>Horizontal Alignment</b>		
Minimum Radius	0 m suspension/cable-stayed spans	Std. Plan R-107-D1
Minimum Length of Curve	NA	
Minimum Radius Not Requiring a Spiral	NA	
Maximum Super elevation	5%	Std. Plan R-107-D1
Maximum Rollover (shoulder)	6%	Std. Plan R-107-D1
<b>Vertical Alignment</b>		
Maximum Percent of Grade	5.0%	AASHTO Chapter 3 (p. 239)
Minimum Percent of Grade	0.3%	AASHTO Chapter 3 (p. 242)
Minimum Stopping Sight Distance	85 m	AASHTO Exhibit 3-1 (p. 112)
Minimum K-Value For Crest VC	11	AASHTO Exhibit 3-76 (p. 274)
Minimum K-Value For Sag VC	18	AASHTO Exhibit 3-79 (p. 280)
Minimum Vertical Clearance over Detroit River	See navigation envelope	US Coast Guard
Minimum Vertical Clearance To Roadways (desirable)	4900 mm min/5000 mm desired	BDM 7.01.08 Desired for New Freeways
Minimum Railroad Vertical Clearance	7010 mm	BDM 13.04.04
Minimum Railroad Horizontal Clearance	6100 mm Crash Barrier required for piers < 7620 mm from track centerline	BDM 13.04.03 BDM 13.04.09

Source: Parsons Transportation Group, 4/15/05

AASHTO = American Association of State Highway and Transportation Officials: A Policy on Geometric Design of Highways and Streets, 2004

BDM= Michigan Bridge Design Manual

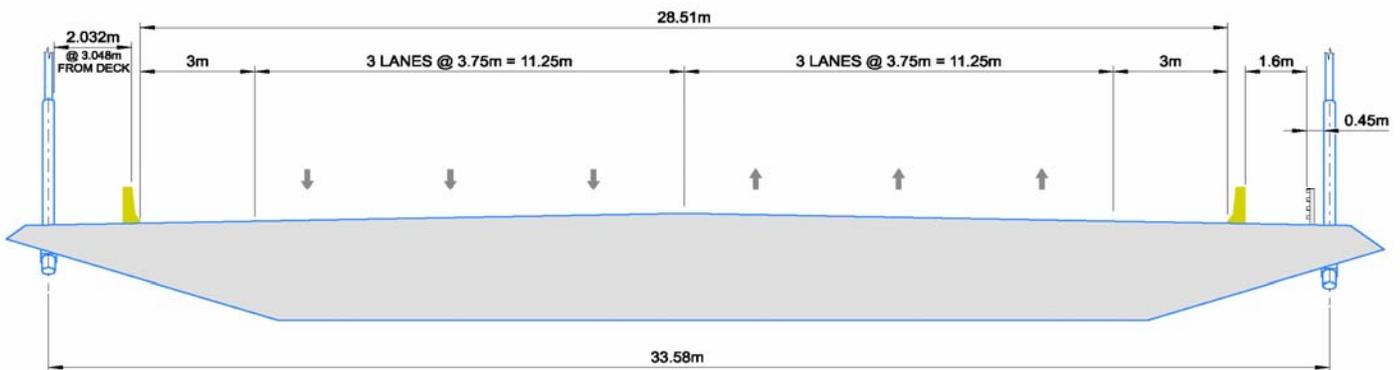
Std Plan = MDOT Standard Plans

## 2.3. Cross Section

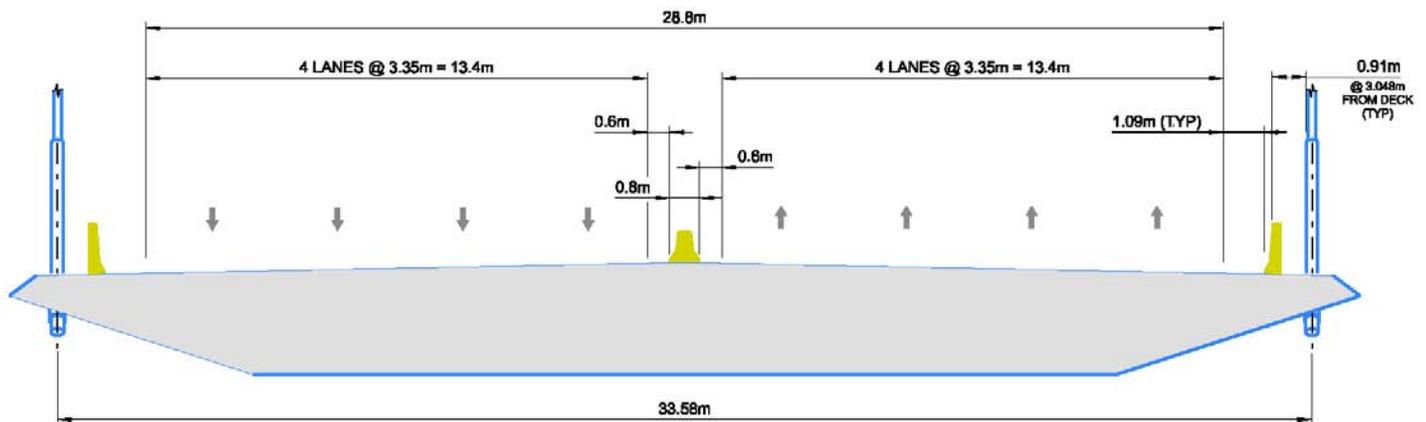
*Table 2*

Item	Criteria	Reference
Total Number of Lanes	3-lanes each direction	Cross Section Tech. Memo 6/29/06
Lane Width	3.75 m	Cross Section Tech. Memo 6/29/06
Right Shoulder Width	3.0 m	Cross Section Tech. Memo 6/29/06
Pavement Cross Slope	2.0% (English BDG)	BDG 6.05.01
Shoulder Cross Slope	2.0% (English BDG)	BDG 6.05.01
ZOI Clearance Box (TL-4 rlg.)	2.032m from top face of railing at 3.048m from deck	Cross Section Tech. Memo 6/29/06
<b>Physical Elements</b>		
Traffic Barrier - Exterior	TL-4	Cross Section Tech. Memo 6/29/06
Traffic Barrier - Median	TL-4 design for future loading only	Cross Section Tech. Memo 6/29/06
Sidewalk	1.6m one side only	Cross Section Tech. Memo 6/29/06
Sidewalk Railing	42" high	

BDG = Michigan Bridge Design Guide



**Figure 1 - Example Cross Section**



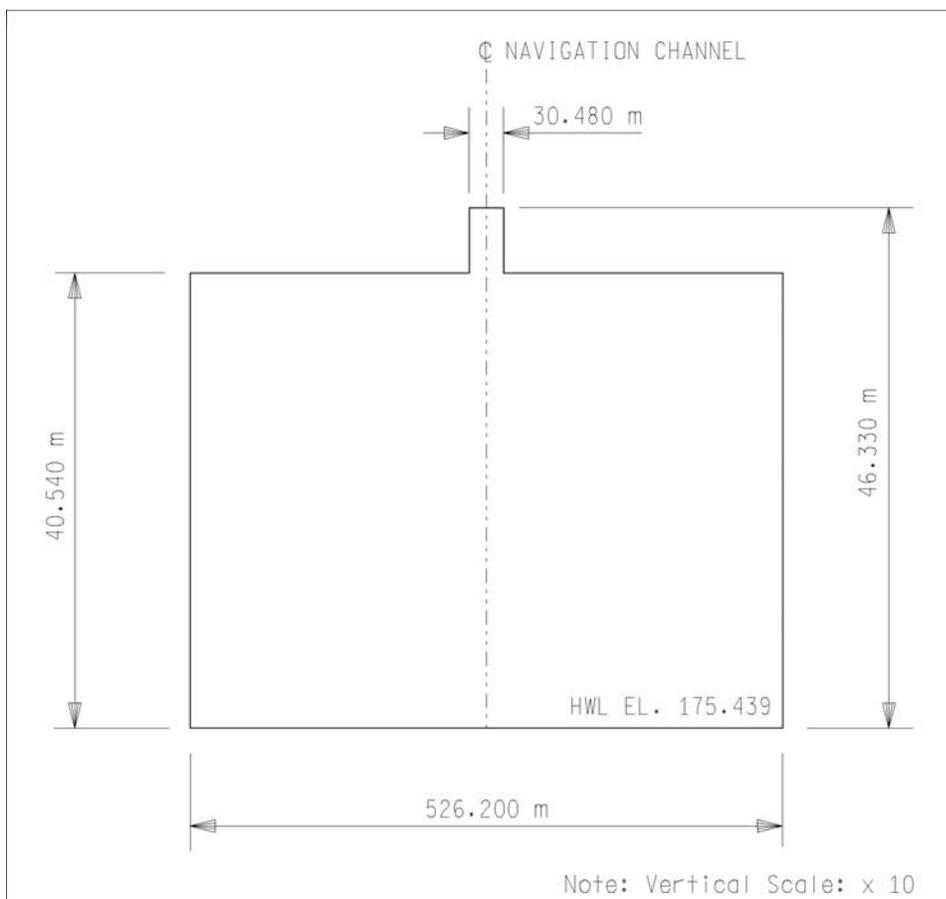
**Figure 2 - Future Cross Section**

The cross sections in **Figures 1 and 2** are shown with a box type superstructure and solid barrier for example only. Actual cross sections will vary for each bridge type to be studied. Variations will include barrier type, structure type, depth, cable configuration, etc. **Figure 1** shall be used for design purposes.

Design allowance shall be made for possible future loading condition shown in **Figure 2**.

## 2.4. Channel Clearances

The minimum horizontal and vertical navigational clearance is shown in **Figure 3**. Horizontal dimensions are measured perpendicular to centerline of the navigation channel.



**Figure 3 – Navigation Envelope**

Profile grade lines will be adjusted for variations in structure depth and allowance for live load deflection.

*River Pier Restrictions* — to be determined.

### **3. DESIGN**

#### **3.1. Design Life**

The design life for the bridge for assessing serviceability shall be 120 years. The design life, for statistical purposes, shall be 75 years in accordance with AASHTO *LRFD Bridge Design Specifications* article 1.2 - Definitions.

#### **3.2. Service Life**

The service life for all bridge components shall be 120 years. For specific components where it is not practical to achieve a 120 year life, then these components shall be designed for replaceability. Examples of such components include, but are not limited to, the following:

- Stay Cables
- Bearings
- Expansion Joints
- Deck Wearing Surface
- Navigation Lighting
- Roadway Lighting

During Conceptual Design, the bridge components requiring replacement shall be identified and included in the life cycle bridge cost evaluation.

#### **3.3. Redundancy**

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

#### **3.4. Operational Importance and Seismic Classification**

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

### **4. DESIGN LOADS AND FORCES**

#### **4.1. Special Loading Conditions**

Design loading at construction: 6 striped traffic lanes with 3m shoulders (seven design traffic lanes), exterior barrier, sidewalk & sidewalk barrier on one side of the structure (Figure 1).

A future loading configuration shall be considered: 8 traffic lanes, exterior TL-5 barrier and TL-4 median barrier, no sidewalk (Figure 2).

#### **4.2. Dead Loads**

Dead loads shall be in conformance with traditional unit weights provided in the AASHTO *LRFD Bridge Design Specifications* and *Canadian Highway Bridge*

*Design Code.* The bridge deck wearing surface shall be designed to be replaceable. No future overlay provision is made. During conceptual design, additional loads will be introduced.

### **4.3. Vertical Live Load**

Consideration shall be given, during Conceptual Design, to the lane loading configurations unique to a border crossing, which frequently result in a concentration of trucks due to long truck queues in one or more lanes. This consideration may result in a special supplementary lane loading.

The design vehicular live load shall be as specified in the *AASHTO LRFD Bridge Design Specifications* and *Canadian Highway Bridge Design Code* for all limit states.

### **4.4. Earthquake Loading**

The bridge shall be classified as a critical structure within Zone 1. For spans less than 100 meters nominal span, no seismic analysis is required and only minimum seat widths and superstructure connection forces shall be considered in the design. For spans greater than 100 meters, structure shall be assessed for seismic loading using either dynamic analysis with appropriate ground motion time histories or a multimodal spectral analysis may be employed with a seismic coefficient of 0.08 g (8% of gravitational acceleration).

### **4.5. Wind Loading**

For the Conceptual Design phase the base wind design velocity as defined in *AASHTO LRFD Bridge Design Specifications* article 3.8.1.1 shall be used. For preliminary design of the preferred alternative a desktop aerodynamic study shall be performed using wind data from nearby sources. During final design a full static and dynamic wind tunnel testing shall be performed.

### **4.6. Thermal Loading**

Design temperature range shall be in conformance with the *MDOT Bridge Design Manual* and *Canadian Highway Bridge Design Code*. During conceptual design, design temperature gradients for deck elements, cables, tower legs, or other elements will be introduced.

### **4.7. Stream Flow / Scour**

During final design, bridge pier scour estimates and forces on bridge piers due to stream flow shall be determined based on a hydraulics analysis. For piers placed in the river the scour analysis shall make use of: FHWA Hydraulic Engineering Circular No. 18, *Evaluating Scour at Bridges*, FHWA-IP-90-017, 3rd Edition, 1995.

FHWA Scour Service load design will provide for the current condition of the Detroit River as well as the full depth of scour from the scour analysis.

#### 4.8. Stay Cable

Stay Cables shall be designed in accordance with PTI *Recommendations for Stay Cable Design, Testing and Installation*. The bridge shall be designed for cable replacement and possible conditions involving accidental loss (or breakage) of any one cable.

#### 4.9. Suspension Cable and Suspenders

Main Cable Wire (presumes air spun parallel wire cable) shall be designed using maximum working load limit (stress) and minimum factor of safety with the following for the Main Cable Wire:

Maximum allowable direct tensile stress: 689 MPa (based on conventional Zinc Coated Bridge Wire).

0.305m minimum bed radius at strand shoes

For suspenders and sockets:

Minimum F.S. = 4.0 for service loads based upon ultimate catalog breaking strength. This factor of safety includes the loss of rope efficiency due to bend over cable bands with a minimum bend diameter 11 times the rope diameter.

The bridge shall be designed for suspender replacement and possible conditions involving accidental loss (or breakage) of any one suspender.

#### 4.10. Group Load Combinations

Loading combinations shall be in conformance with AASHTO *LRFD Bridge Design Specifications*, *Canadian Highway Bridge Design Code* and usual practice. During conceptual design, load combinations and load factors for extreme events, for elements such as stay cables or suspenders, or for special conditions will be introduced.

#### 4.11. Vessel Collision Forces

Vessel impact loading shall be for a critical bridge in conformance with the AASHTO *LRFD Bridge Design Specifications* and *Canadian Highway Bridge Design Code*. During final design, vessel impact forces applied to piers in the waterway are to be determined by a vessel impact study.

#### 4.12. Ice Loads

Structural ice loads shall be in conformance with AASHTO *LRFD Bridge Design Specifications*, *Canadian Highway Bridge Design Code*.

### 5. SAFETY AND SECURITY

1. The permanent works shall be sufficiently robust to restrict to acceptable levels their vulnerability to accidental or malicious damage.
2. Security against unauthorized access to the various parts of the permanent works shall be provided with details as appropriate to each individual area.

3. Additional security measures to be developed in conjunction with the owner in final design.

## **6. MATERIALS**

Materials, including concrete, mild steel reinforcing, prestressing steel, and structural steel shall be in conformance with the MDOT *Bridge Design Manual*. During conceptual design, additional standards, codes, and specifications for material properties will be introduced.

TYPE OF PROJECT: Structural HWY NO: 401  
 LENGTH: TBD

LOCATION: City of Windsor

COUNTY OF: Essex  
 TOWNSHIP OF: Sandwich West

	PRESENT CONDITIONS	DESIGN STANDARDS	PROPOSED STANDARDS
HIGHWAY CLASSIFICATION		UAD 80	UAD 80
MIN STOPPING SIGHT DIST		135 m	135 m
EQUIVALENT MIN 'K' FACTOR		Crest – 35 Sag – 30	Crest - 35 Sag - 30
GRADES MAXIMUM		6 – 8%	5%
MINIMUM RADIUS		250 m	400 m (a) (b)
PAVEMENT WIDTH		6 x 3.75 m	6 x 3.75 m
SHOULDER WIDTH		3.00 m	3.00 m
SHOULDER ROUNDING		1.00 m	n/a (c)
MEDIAN WIDTH		1.00 m	1.00 m
R.O.W. WIDTH		40 m	TBD
POSTED SPEED		60 km/h	60 km/h

TYPE OF PROJECT: Structural HWY NO: 401  
 LENGTH: TBD

LOCATION: City of Windsor

**TRAFFIC DATA:**

Location: Hwy	Distance Km	2015 AADT	2025 AADT	2035 AADT	% DHV	% COMM	C.R.
New International Crossing	TBD	28,200	34,200	39,100	6.5	43	n/a

\* 2000 Provincial Average Collision Rate is 0.6. (Rate is the number of accidents per million vehicle kilometres of travel (MVKM))

**Notes:**

- a) The minimum horizontal curve proposed in Canada has a radius of 400 m. A horizontal curve with a radius of 250 m is proposed in the U.S.
- b) Horizontal curve radius pertains to the bridge approach. The main crossing structure is on tangent.
- c) Shoulder rounding is not applicable to structures.

**Remarks:**

1) Scope of Work

The Border Transportation Partnership, which is comprised of the Ontario Ministry of Transportation (MTO), Transport Canada, the Michigan Department of Transportation and the Federal Highway Administration, is undertaking an Environmental Assessment (EA) study for the Detroit River International Crossing (DRIC). This EA study will identify the location for a new river crossing, plazas for border inspections and connecting roads leading from Highway 401 in Canada to the Interstate Highway system in the U.S.

This Design Criteria has been written for the new river crossing structure only. A subsequent Design Criteria will be prepared for the connecting roads leading from Highway 401 to the plaza for border inspections.

2) Limits of Project

The international crossing location is to be selected within an 'Area of Continued Analysis' developed as part of the DRIC study (see attached key map).

3) Adjacent Projects/History

*Planning/Needs and Feasibility Study* - The Border Transportation Partnership conducted a Planning/Needs and Feasibility (PN/F) Study to develop a long term transportation strategy that would ensure the safe and efficient movement of people, goods and services across the United States and Canadian border within the region of Southeast Michigan and Southwest Ontario, including improved connections to national, provincial, and regional transportation systems. The PN/F study was completed in January 2004.

*Ontario Environmental Assessment (OEA) Terms of Reference* – After the completion of the PN/F and as required under the OEA Act, MTO developed a Terms of Reference for the preparation of an Environmental Assessment and submitted it to the Ontario Minister of the Environment for review in May 2004. The TOR was approved in September 2004.

TYPE OF PROJECT: Structural HWY NO: 401  
LENGTH: TBD

LOCATION: City of Windsor

4) Proposed Design Standards

Design standards were developed using the Geometric Design Standards for Ontario Highways Manual and a study of other international crossing structures in Southern Ontario.

5) Construction Staging

Construction staging will be developed during the later stages of the project.

6) Property

Property acquisition will be required for the construction of the international crossing. Property requirements will be determined during the later stages of the project.

7) Illumination

Full illumination of the international crossing structure is proposed.

8) Traffic Signals

Traffic signals are not proposed on the international crossing.

9) Traffic Counting Stations

The need for traffic counting stations will be investigated during the later stages of the project.

10) Traffic Barriers and Roadside Safety

Given the high percentage of commercial vehicles projected to travel on the new international crossing, PL-3 (TL-5) exterior concrete barrier is proposed.

11) Sidewalks and Bike Paths

Provisions for one sidewalk and bike paths (shoulders) have been included on the international crossing.

12) Private/Commercial Entrances

Private and commercial entrances will not be permitted to directly access the approaches to the international crossing.

13) Railways

The Essex Terminal Railway (ETR), consisting of a mainline and several spur lines, operates within the ACA and in the area of the proposed international crossing. Any crossing of the ETR will be grade separated.

14) Utilities

- Hydro
- Gas
- Cable
- Telephone

TYPE OF PROJECT: Structural HWY NO: 401  
LENGTH: TBD

LOCATION: City of Windsor

15) Pipe Lines

A high pressure gas pipeline operated by British Petroleum Canada (BP) is located within the ACA and crosses the Detroit River north of Prospect Avenue on the Windsor Salt Company site.

16) Municipal Drains

Location of municipal drains to be determined in consultation with the City of Windsor and Essex Region Conservation Authority.

17) Drainage

Drainage design will be completed during the later stages of the project.

18) Signing

Directional and regulatory signage as well as signage for traffic streaming (FAST, NEXUS, etc.) will be provided on the international crossing structure.

19) Foundation Investigation

Foundations Investigations are being undertaken to better understand the effects of solution mining of salt deposits and to confirm the integrity of the underlying bedrock to support a new international bridge spanning the Detroit River.

The first part of the foundations investigations program includes drilling 12 boreholes to a depth of 500 m in the vicinity of Practical Alternative Crossing B and C alignments due to the existence of brine wells from historical salt mining activities in the area. The drilling of boreholes is not proposed along Crossing A as this alignment is sufficiently removed from areas of solution mining. A similar drilling program is being undertaken on the U.S. side of the river.

The second part of the investigations includes geophysical testing. Once drilling has been completed and the borehole casings installed, the ground between boreholes will be characterized using cross-hole seismic tomography.

A Geoadvisory Group has been assembled to assist the study team in completing the foundations investigations program. The group is comprised of geotechnical experts from Canada and the United States. The results of the drilling program, including seismic tomography, will be reviewed by the group and will be used in the evaluation process for selecting the preferred alternative of the new international bridge crossing.

20) Connecting Links

No connecting link is present in the vicinity of the proposed international crossing.

21) Assumptions, Designations, Transfers and Road Closings

- Assumptions – n/a
- Designations – n/a
- Transfers – n/a
- Road Closings – TBD

TYPE OF PROJECT: Structural \_\_\_\_\_ HWY NO: 401  
LENGTH: TBD \_\_\_\_\_

LOCATION: City of Windsor

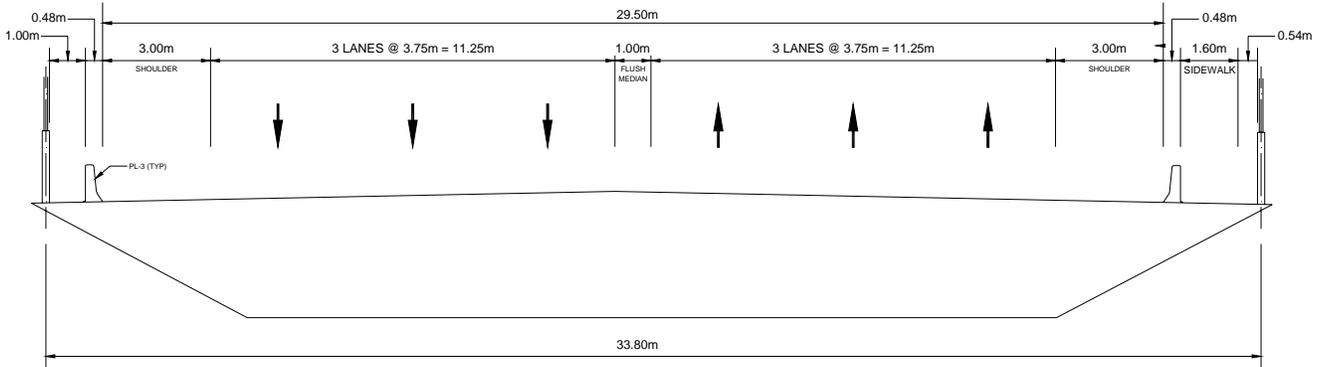
22) Environmental Assessment Report

An environmental assessment report will be prepared as part of the environmental assessment process.

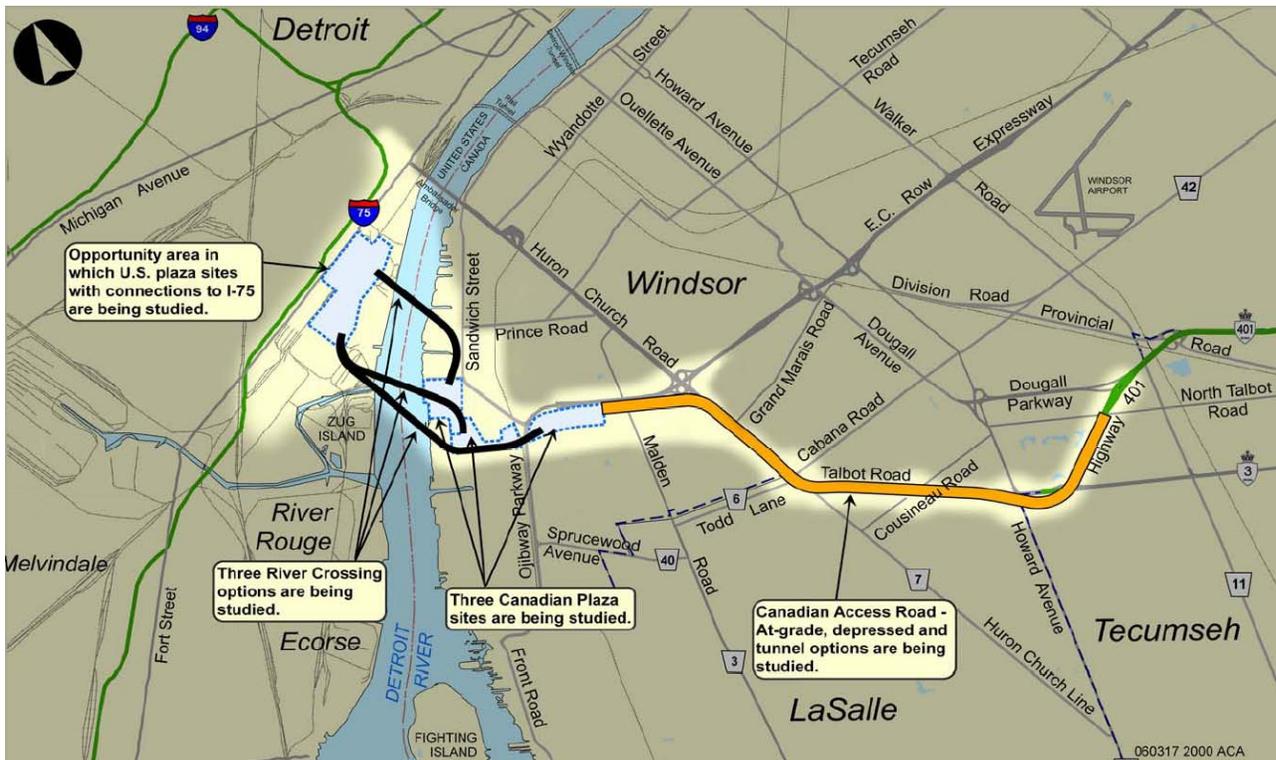
TYPE OF PROJECT: Structural HWY NO: 401  
 LENGTH: TBD

LOCATION: City of Windsor

**Typical Section**



**Key Map**



## **Appendix C: Cost Estimate Summary**

Preliminary

Project DRIC  
 By BLC  
 Chk SHC  
 File: F:\646294\_DRIC\_Study\01.0000 Alternative Development\01.0200 Practical Alternatives\01.0210 Alternative Development\01.0211 Bridge\Cost Estimates\Bridge Cost Summary 070130.xls

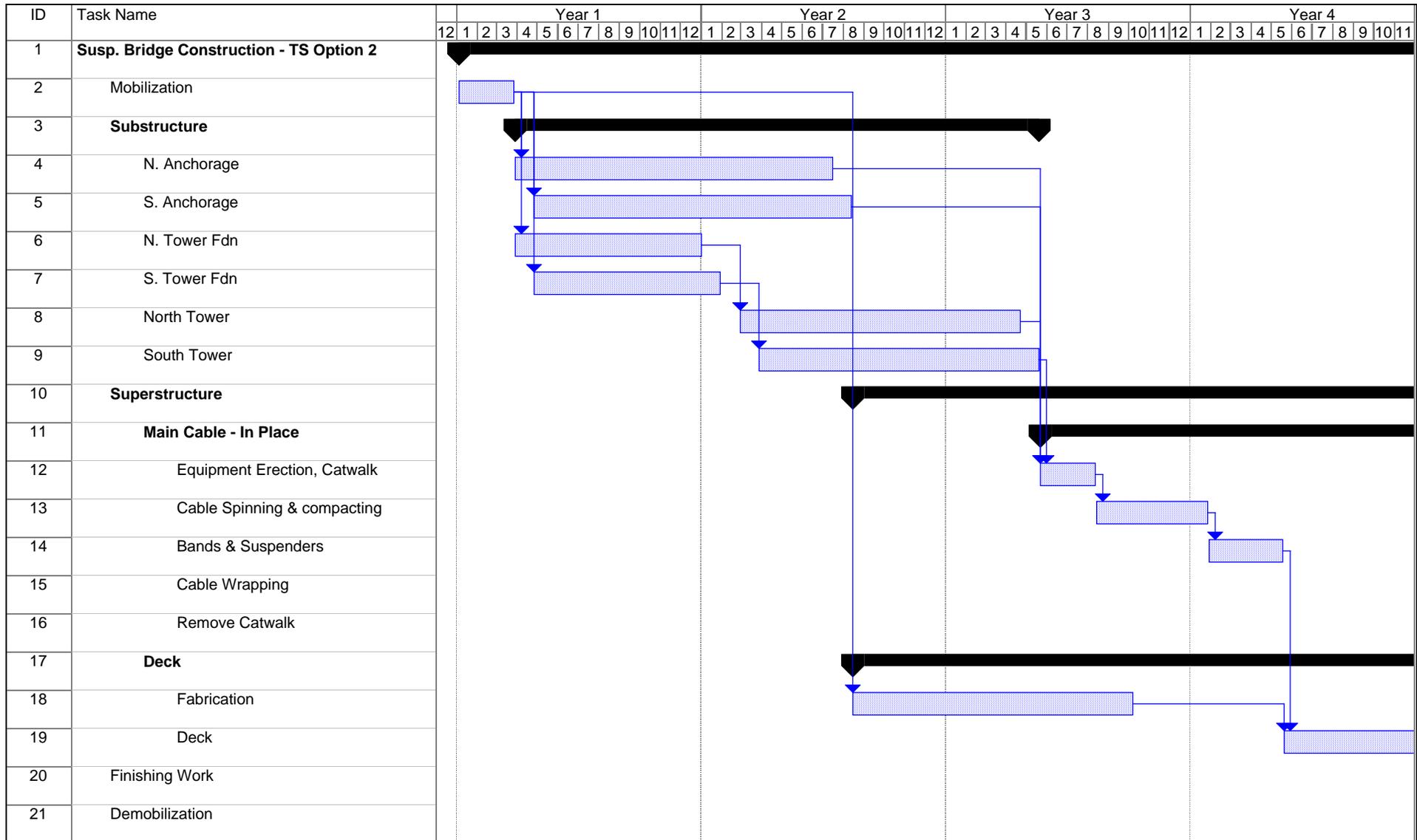
Job No. 646294  
 Subject Summary Bridge Cost Estimate

**Type Study Construction Cost Estimate Summary**

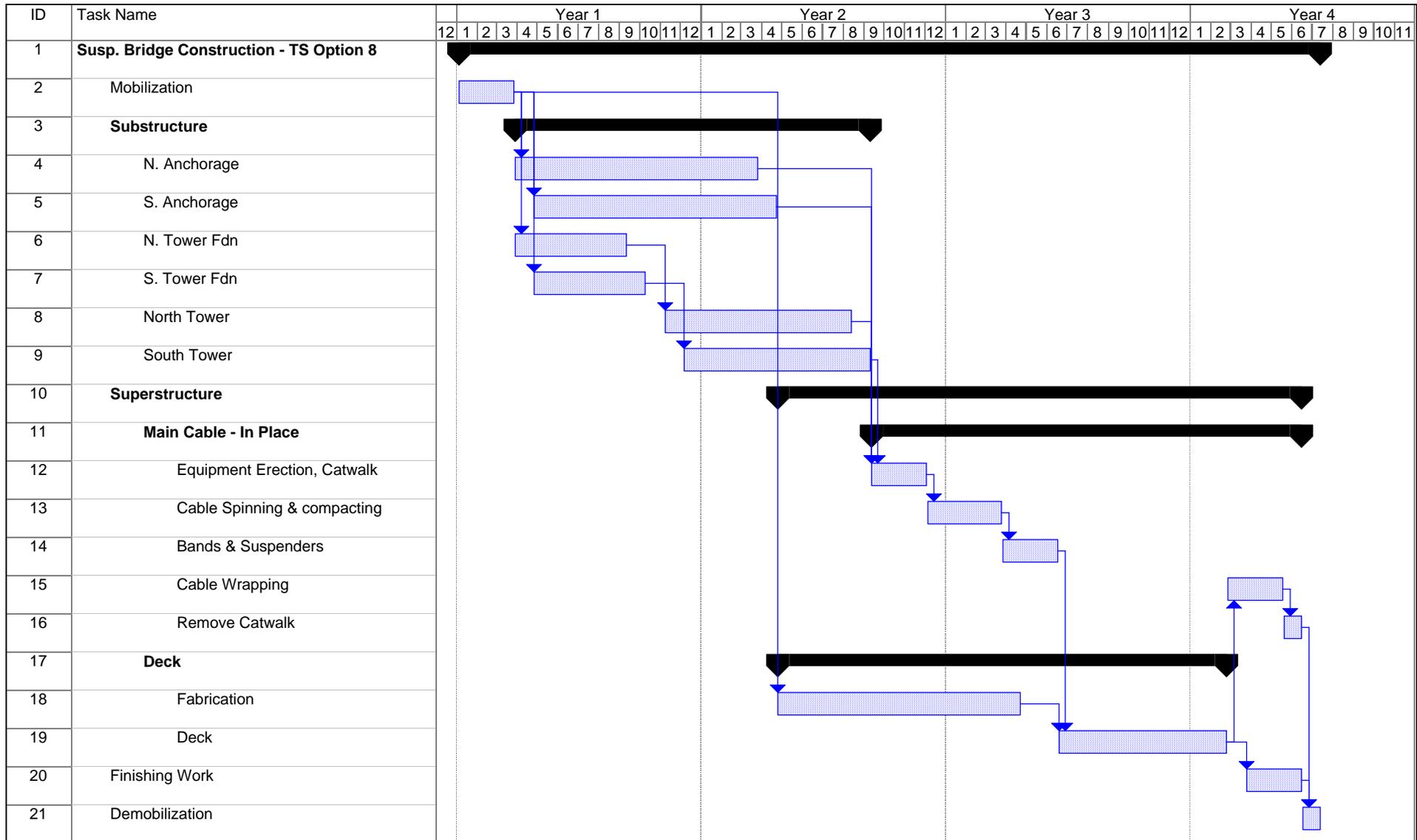
Geometry	Type Study Option										
	1	2	3	4	5	6	7	8	9	10	11
	X10(A)			X10(B)				X11(C)			
Main Span Length (m)	1,300	925	925	860	600	870	870	600	750	750	750
Suspended Spans Length (m)	1,300	1,300	1,757	1,460	1,164	1,250	870	1,092	1,476	750	1,250
Approaches (m)	2,701	2,672	2,215	1,024	1,195	1,234	1,614	1,267	1,543	2,299	1,769
Total Bridge Length	4,001	3,972	3,972	2,484	2,359	2,484	2,484	2,359	3,019	3,049	3,019
<b>Cost Estimate (2006 US\$)</b>	<b>1</b>	<b>2</b>	<b>3</b>	<b>4</b>	<b>5</b>	<b>6</b>	<b>7</b>	<b>8</b>	<b>9</b>	<b>10</b>	<b>11</b>
<b>Main Bridge</b>	Suspension (UB)	Suspension (UB)	Cable Stay	Cable Stay	Cable Stay	Suspension	Suspension (UB)	Suspension	Cable Stay	Suspension (UB)	Suspension
Superstructure											
Deck	88,800,000	89,156,000	151,416,400	124,998,200	95,013,500	87,400,000	59,654,400	77,111,100	123,159,000	51,840,000	87,400,000
Suspension System	103,281,500	73,998,000	48,017,800	35,999,000	24,564,100	56,611,100	55,680,200	39,392,100	36,596,400	41,852,500	52,694,500
Subtotal	192,081,500	163,154,000	199,434,200	160,997,200	119,577,600	144,011,100	115,334,600	116,503,200	159,755,400	93,692,500	140,094,500
Substructure											
Tower/Pylon	29,090,700	20,564,100	35,582,000	32,340,500	22,139,800	16,121,700	16,121,700	11,013,500	27,892,800	13,255,600	16,569,500
Tower/Pylon Foundation	18,380,500	18,380,500	21,891,200	18,380,500	11,688,700	18,380,500	18,380,500	10,945,600	14,663,000	14,663,000	14,663,000
Anchorage/Anchor Piers	141,838,900	85,790,200	4,222,000	16,887,700	2,907,200	91,433,900	91,433,900	63,622,500	3,624,000	93,231,000	93,231,000
Subtotal	189,310,100	124,734,800	61,695,200	67,608,700	36,735,700	125,936,100	125,936,100	85,581,600	46,179,800	121,149,600	124,463,500
Marine Construction	-	24,500,000	24,500,000	-	22,400,000	-	-	22,400,000	-	-	-
Subtotal	381,391,600	312,388,800	285,629,400	228,605,900	178,713,300	269,947,200	241,270,700	224,484,800	205,935,200	214,842,100	264,558,000
Miscellaneous Items	28,183,100	28,183,100	38,090,500	31,651,800	25,234,700	27,099,100	18,861,000	23,673,800	31,998,600	16,259,500	27,099,100
Quantities Subtotal (rounded)	410,000,000	341,000,000	324,000,000	260,000,000	204,000,000	297,000,000	260,000,000	248,000,000	238,000,000	231,000,000	292,000,000
Mobilization 5%	20,500,000	17,050,000	16,200,000	13,000,000	10,200,000	14,850,000	13,000,000	12,400,000	11,900,000	11,550,000	14,600,000
Max. Design Contingency <sup>1</sup> 10-20%	20%	20%	20%	20%	20%	10%	10%	10%	20%	10%	10%
Max. Design Contingency <sup>1</sup>	86,100,000	71,610,000	68,040,000	54,600,000	42,840,000	31,185,000	27,300,000	26,040,000	49,980,000	24,255,000	30,660,000
Min. Design Contingency <sup>1</sup> 5-10%	10%	10%	10%	10%	10%	5%	5%	5%	10%	5%	5%
Min. Design Contingency <sup>1</sup>	43,050,000	35,805,000	34,020,000	27,300,000	21,420,000	15,592,500	13,650,000	13,020,000	24,990,000	12,127,500	15,330,000
Max. Construction Contingency <sup>2</sup> 20%	103,320,000	85,932,000	81,648,000	65,520,000	51,408,000	68,607,000	60,060,000	57,288,000	59,976,000	53,361,000	67,452,000
Min. Construction Contingency <sup>2</sup> 10%	47,355,000	39,385,500	37,422,000	30,030,000	23,562,000	32,744,250	28,665,000	27,342,000	27,489,000	25,467,750	32,193,000
<b>Max. Main Bridge Total (rounded)</b>	<b>620,000,000</b>	<b>516,000,000</b>	<b>490,000,000</b>	<b>393,000,000</b>	<b>308,000,000</b>	<b>412,000,000</b>	<b>360,000,000</b>	<b>344,000,000</b>	<b>360,000,000</b>	<b>320,000,000</b>	<b>405,000,000</b>
<b>Min. Main Bridge Total (rounded)</b>	<b>521,000,000</b>	<b>433,000,000</b>	<b>412,000,000</b>	<b>330,000,000</b>	<b>259,000,000</b>	<b>360,000,000</b>	<b>315,000,000</b>	<b>301,000,000</b>	<b>302,000,000</b>	<b>280,000,000</b>	<b>354,000,000</b>
<b>Approach Bridge<sup>4</sup></b>											
Bridge	190,629,000	188,606,000	156,351,000	72,292,000	84,373,000	87,114,000	113,935,000	89,455,000	108,875,000	162,289,000	124,826,000
Mobilization 5%	9,531,450	9,430,300	7,817,550	3,614,600	4,218,650	4,355,700	5,696,750	4,472,750	5,443,750	8,114,450	6,241,300
Max. Design Contingency <sup>1</sup> 25%	50,040,113	49,509,075	41,042,138	18,976,650	22,147,913	22,867,425	29,907,938	23,481,938	28,579,688	42,600,863	32,766,825
Min. Design Contingency <sup>1</sup> 15%	30,024,068	29,705,445	24,625,283	11,385,990	13,288,748	13,720,455	17,944,763	14,089,163	17,147,813	25,560,518	19,660,095
Max. Construction Contingency <sup>2</sup> 20%	50,040,113	49,509,075	41,042,138	18,976,650	22,147,913	22,867,425	29,907,938	23,481,938	28,579,688	42,600,863	32,766,825
Min. Construction Contingency <sup>2</sup> 10%	23,018,452	22,774,175	18,879,383	8,729,259	10,188,040	10,519,016	13,757,651	10,801,691	13,146,656	19,596,397	15,072,740
<b>Max. Appr. Bridge Total (rounded)</b>	<b>300,000,000</b>	<b>297,000,000</b>	<b>246,000,000</b>	<b>114,000,000</b>	<b>133,000,000</b>	<b>137,000,000</b>	<b>179,000,000</b>	<b>141,000,000</b>	<b>171,000,000</b>	<b>256,000,000</b>	<b>197,000,000</b>
<b>Min. Appr. Bridge Total (rounded)</b>	<b>253,000,000</b>	<b>251,000,000</b>	<b>208,000,000</b>	<b>96,000,000</b>	<b>112,000,000</b>	<b>116,000,000</b>	<b>151,000,000</b>	<b>119,000,000</b>	<b>145,000,000</b>	<b>216,000,000</b>	<b>166,000,000</b>
<b>Max. Grand Total (rounded)<sup>5</sup></b>	<b>920,000,000</b>	<b>810,000,000</b>	<b>740,000,000</b>	<b>510,000,000</b>	<b>440,000,000</b>	<b>550,000,000</b>	<b>540,000,000</b>	<b>490,000,000</b>	<b>530,000,000</b>	<b>580,000,000</b>	<b>600,000,000</b>
<b>Min. Grand Total (rounded)<sup>5</sup></b>	<b>770,000,000</b>	<b>680,000,000</b>	<b>620,000,000</b>	<b>430,000,000</b>	<b>370,000,000</b>	<b>480,000,000</b>	<b>470,000,000</b>	<b>420,000,000</b>	<b>450,000,000</b>	<b>500,000,000</b>	<b>520,000,000</b>
Max. Main Bridge Cost/m <sup>2</sup>	\$ 14,203	\$ 11,820	\$ 8,305	\$ 8,016	\$ 7,880	\$ 9,815	\$ 12,323	\$ 9,381	\$ 7,263	\$ 12,706	\$ 9,649
Min. Main Bridge Cost/m <sup>3</sup>	\$ 11,935	\$ 9,919	\$ 6,983	\$ 6,731	\$ 6,626	\$ 8,577	\$ 10,782	\$ 8,208	\$ 6,093	\$ 11,118	\$ 8,434
Max. Main Bridge Cost/ft <sup>2</sup>	\$ 1,319	\$ 1,098	\$ 772	\$ 745	\$ 732	\$ 912	\$ 1,145	\$ 872	\$ 675	\$ 1,180	\$ 896

- Notes:
- Design contingency reflects the level of design completed for this particular phase of the project. The design contingency may also differ between components (e.g., the approach bridge design contingency is greater due to a lower level of design such as potential changes in the geometry of the approaches as the Plaza design progresses) and structure types.
  - Construction Contingency is a factor to cover risk and uncertainty in the construction of the project from factors such as material price volatility, unforeseen site conditions, environmental mitigation, etc. This factor does NOT include a management contingency or reserve for third party or unanticipated changes. Source: <http://www.fhwa.dot.gov/programadmin/mega/contingency.htm>
  - Cost estimates do NOT include soft costs such as engineering or inflation.
  - Crossing X-10(A) - US Plaza 4 to Canadian Plaza A  
 Crossing X-10(B) - US Plaza 4 to Canadian Plaza B1  
 Crossing X-11(C) - US Plaza 6 to Canadian Plaza B (Crossings to Plaza C reduce costs approximately \$20 million)
  - Maximum and Minimum costs are based on variations in contingency percentages in order to realistically portray a structure cost range given the estimating methodology.

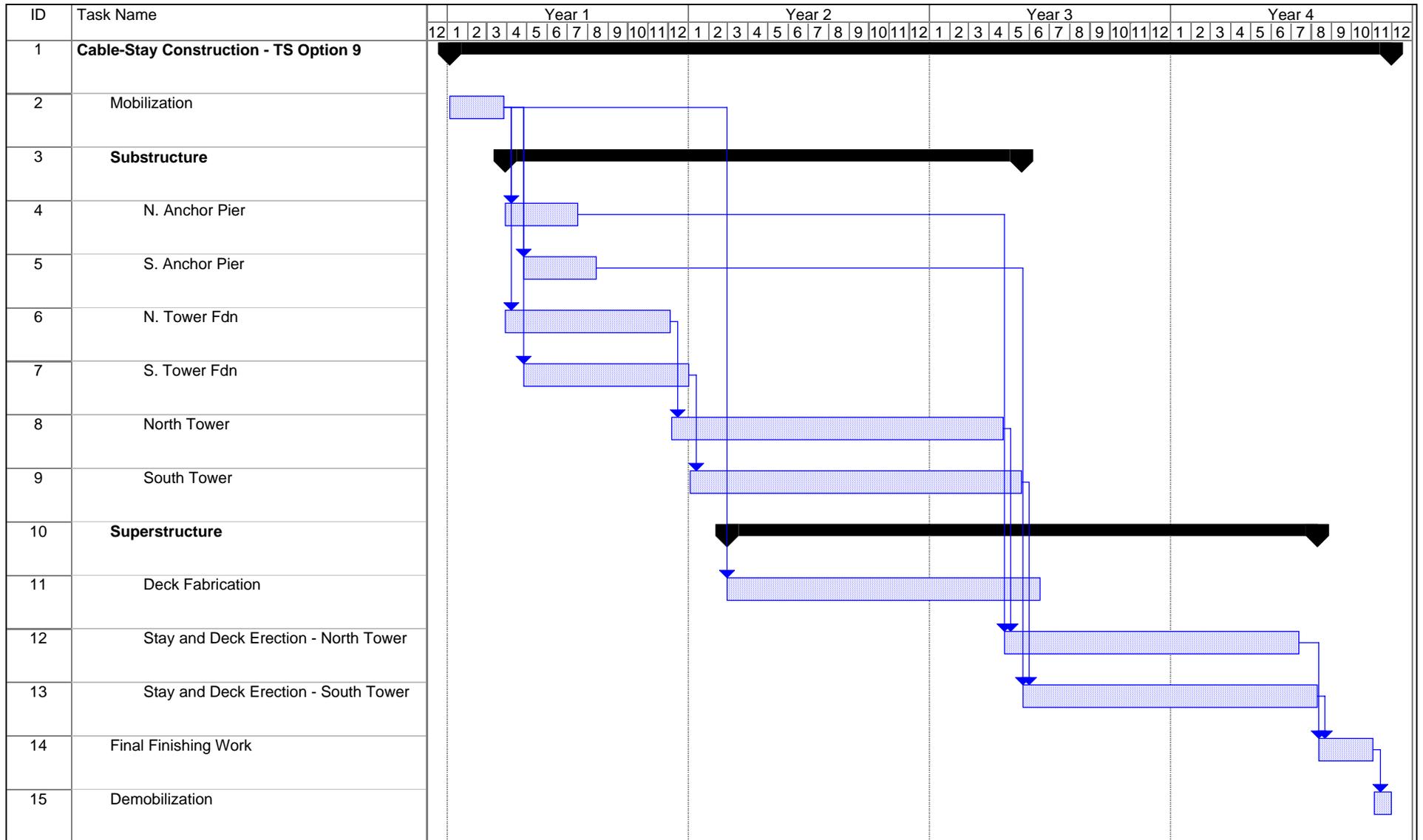
## **Appendix D: Representative Construction Schedules**



Project: DRIC Susp. Bridge Const. Date: Thu 1/25/07	Task		Milestone		External Tasks	
	Split		Summary		External Milestone	
	Progress		Project Summary		Deadline	



Project: DRIC Susp. Bridge Const. Date: Thu 1/25/07	Task		Milestone		External Tasks	
	Split		Summary		External Milestone	
	Progress		Project Summary		Deadline	



Project: Suspension Bridge TS#1 v1 r'  
Date: Thu 1/25/07

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

## **Appendix E: Geotechnical Report**

*Draft*

**Preliminary Draft Geotechnical Evaluation Report  
Crossings X-10 and X-11  
United States Shoreline**

**Proposed Detroit River International Crossing  
Proposed Foundation Elements**

**September 11, 2006**

**Revised September 21, 2006**

**Prepared by:**

**NTH Consultants, Ltd.**

**Prepared for:**

**The Corradino Group  
In Partnership with  
Parsons Transportation Group**

## **1.0 SUMMARY**

The Michigan Department of Transportation (MDOT) has identified the need for a new crossing of the Detroit River between Detroit, Michigan and Ontario, Canada. At this time, the project team has selected two potential crossing corridors, defined as Crossings X-10 and X-11 as shown on the attached Figure No. 1.

The initial geotechnical task performed by NTH Consultants, Ltd. (NTH) for this project (Task 2330) was to study the Illustrative Alternative crossing locations, including collecting the relevant available geotechnical data along the proposed project area and evaluating the data with respect to conceptual designs. The results of this task were presented in our report entitled Geotechnical Evaluation, Proposed Detroit River International Crossing, Task 2330, dated December 28, 2005. The purpose of the current paper is to summarize the historical data specifically relevant to proposed crossings corridors X-10 and X-11, provide preliminary cross sections of the river crossing corridors, and provide a summary of expected design and construction issues for the specific crossing locations.

All elevations presented are based on USGS datum and dimensions are in meters (feet). All interpretations are for United States (US) side only.

## **2.0 CROSSING DESCRIPTIONS**

The two subject crossings are in the same general vicinity, between the Ambassador Bridge, and Zug Island in southwest Detroit. These are described as follows:

### **2.1 X-10 CROSSING CORRIDOR**

The Crossing X-10 corridor generally consists of the area immediately north of Zug Island to historic Fort Wayne along the banks of the Detroit River. The area is generally flat with a slight drop in elevation at the river, with large vacated areas, parking lots, and paved/unpaved roads. Current land use includes light to moderate industrial areas,

including a cement terminal operation, a major trucking terminal, truck ferry operation, and aggregate storage areas. Residential areas exist north of Jefferson Avenue, but are generally intermingled with light commercial and industrial areas. Historic land use includes light to heavy industrial areas, including a major chemical processing plant and power plant operations, along with two suspected solution well operations. Known solution mining wells exist adjacent to the Rouge River, along the south portion of the corridor, as well as possible undocumented solution mining wells adjacent to the current Fort Wayne property. Historic maps also indicate the original shoreline of the Detroit River to be set back approximately 5 to 25 meters (16 to 80 feet) from its current position, with possible docks and former boat slips prevalent throughout.

## **2.2 X-11 CROSSING CORRIDOR**

The Crossing X-11 corridor generally consists of the area along the banks of the Detroit River immediately north of historic Fort Wayne to the existing Mistersky Power Plant. The area is generally flat with a slight drop in elevation at the river, with large vacated areas between the river and Jefferson Avenue. Current land use includes light to moderate industrial regions, power generation facilities, and a large vacant lot adjacent to the river. Residential areas exist north of Jefferson Avenue, but are generally intermingled with light commercial and industrial areas. Historic land use includes light to heavy industrial areas, including a major copper and brass fabrication operation, along with two suspected solution well operations. Historic maps indicate the potential solution mining operations exist directly to the north of the historic copper and brass fabrication facility and the northern portion of the corridor, in what is now intermingled residential and commercial areas. Historic maps also indicate the original shoreline of the Detroit River in the X-11 area to be set back approximately 3 to 15 meters (10 to 50 feet) from its current position, with possible docks and former boat slips prevalent throughout.

In addition to the environmental issues, a Detroit Water and Sewerage Department (DWSD) triple barrel outfall sewer runs from a 4.9 meter (16-foot) diameter interceptor sewer under Jefferson Avenue to the Detroit River, extending through the center of the

former copper and brass works property. The outfall sewer in this section is approximately 1.8 m (6 feet) tall, 5.5 m (18 feet) wide, 365 m (1,200 feet) long and the average depth is approximately 3.0 m (10 feet) below ground. Piling supports the last third of the outfall before the river. The current easement alignment, together with the size and shallow depth of the outfall limit potential development opportunities for the site.

### **3.0 REGIONAL GEOLOGIC SUMMARY**

The generalized subsurface geology for the area is summarized as follows:

#### **3.1 SUMMARY OF OVERBURDEN INFORMATION**

The bedrock along the project corridor is overlain by glacially deposited soils (drift), which have been deposited either directly by glacial ice (till), by glacial meltwater streams (glaciofluvial), or by glacial lakes (lacustrine deposits). The upper soil formations along the alignment generally consist of a relatively thick mantle of Wisconsin aged lacustrine clays (10,000 to 50,000 years ago) that, with the exception of the near-surface deposits, are typically medium to stiff in consistency. The lacustrine soils were deposited as sediments from a series of glacial lakes impounded between the ice front and the Inner Defiance Moraine located near the northwest corner of Wayne County. The upper 3 to 6 m (10 to 20 feet) of these deposits have been desiccated during historical low water periods, resulting in soils of very stiff to hard consistency near the surface. The clay soils frequently contain intermittent sand and gravel layers that were produced from glacial rivers carrying coarser sediments as lake levels fluctuated. Localized alluvial soils are present along existing rivers and streams that drain the inland areas. In some locations, lake shorelines are identified by relatively thick layers of sand and gravel.

The lacustrine deposits are typically underlain by a thin layer of highly over-consolidated glacial till, generally consisting of sand, silt, and gravel within a matrix of clay. This

formation is locally termed “hardpan” and usually overlies the bedrock formation. Depending on the amount of clay binder contained in the hardpan, the material may range in nature from cohesive to granular. The hardpan is generally believed to be from the Illinoian Ice age (200,000 years ago) and can also contain calcium carbonate producing a cemented condition. Given the glacial origins of the hardpan layer, occasional cobbles and large boulders are typically present in this layer.

The total drift along the X-10 and X-11 corridors varies in thickness from approximately 27 to 30 meters (90 to 100 feet). The surface topography was formed during the Wisconsin stage (youngest) of Pleistocene Series glaciations of the Cenozoic Era, and has been somewhat modified by surface erosion since that time.

### **3.2 SUMMARY OF BEDROCK INFORMATION**

The proposed crossing corridor is located at the geologically termed southeast margin of the Michigan Basin and within the Erie-Huron lowland. The Michigan Basin is termed as such due to the structural basin shape of the bedrock, in which layers of Paleozoic era sedimentary rock that overlay the Precambrian Basement Complex, dip inwards to the center of the Lower Peninsula from each direction as a series of bowls. The youngest layers of bedrock are located in the center of the state, with older rock layers progressing outwards to the outer margins. Lowland areas occur where the bedrock surface is relatively low compared to other areas of the basin. The Michigan Basin was formed during the late Ordovician Period, when the Taconic Orogeny occurred on the east coast of the United States. The effects of this event caused the structural deformation and localized downward movement in what had been a relatively stable interior continental region.

As a result, several intracratonic structural basins were formed throughout the central lowland areas of North America forming arches and domes. The Michigan Basin is bounded on the west by the Wisconsin Arch and Wisconsin Dome; on the north and northeast by the Canadian Shield; on the east and southeast by the Algonquin Arch in

Ontario and the Findlay Arch in Ohio; and by the Kankakee Arch in northern Indiana and Illinois.

The Michigan Basin has undergone several periods of subsidence and rebound during the Paleozoic Era, creating a complex interbedding of various sedimentary rocks. Barrier reefs formed around the margins of the basin during the Silurian and early Devonian Periods. Carbonate deposits of limestone and dolomite were deposited during the Middle Epoch of the Devonian Period of the Paleozoic Era in shallow salty seas. During periods of marine regression and transgression, anhydrite, gypsum, and salts were precipitated into the basin. During the late Devonian / Early Mississippian Periods, the Acadian Orogeny supplied clastic sedimentary sediments which were eroded from the emerging ancestral Appalachian Mountains and deposited them in intracratonic basins to form sandstones, siltstones, and shale.

Based on the position of Detroit, Michigan, along the southeast rim of the Michigan Basin, the Paleozoic rocks that compromise the basin in this area typically dip to the northwest, with each formation being buried by successive younger formations in the direction of the dip. The regional dip is slight, and is estimated at approximately 6 to 10 meters per kilometer (30 to 50 feet per mile).

The topography of the bedrock surface within the area is somewhat variable and characterized by numerous irregular features in the bedrock surface. The features are believed to have developed before the Pleistocene Epoch and subsequently were modified by repetitive glacial action. The bedrock features include the existence of ancient stream valleys that cut the bedrock surface. Based on historical information, the bedrock features are understood to be fairly broad, and become narrow as they reach the terminus of the Erie / Huron Lowlands.

Due to the movement of the Earth's crust, these strata are seamed and fissured with vertical joints that permit movement of ground water. Where carbon dioxide dissolved

within these groundwater filled cracks, solution voids typically developed within the limestone. Both the limestone and dolomite formations are known to contain dissolved sulfides, which can produce hydrogen sulfide gas upon exposure to atmospheric conditions. Hydrogen sulfide gas in the Detroit area has a history of causing nuisances, and toxic conditions during tunneling operations and deep excavations, causing great bodily harm and even death to construction workers. The natural decay of organic compounds that also existed within the ancient seas became trapped within cavities formed in the limestone and dolomites and is evident today as petroleum. Small amounts of petroleum found within the limestone and dolomite tends to cause discoloring, staining, and produce associative odors.

### **3.3 REGIONAL SEISMOLOGY**

According to historical Seismic risk maps published by the United States Geodetic Survey, Michigan is located within Seismic Risk Zone No. 1 and, as such, poses a relatively low risk for earthquake occurrence. While tremors from earthquakes with epicenters in other regions have been recorded in Michigan, only 34 earthquakes with epicenters in Michigan have been recorded since 1872. With the exception of two seismic events that occurred in the Keweenaw Peninsula at the turn of the 20<sup>th</sup> century, all recorded events had recorded intensities of less than IV on the modified Mercalli scale. This corresponds to approximately magnitude 4.7 on the Richter scale.

According the Geologic Survey Division of the Michigan Department of Environmental Quality, the majority of the above referenced seismic events were the result of slippage along deep-seated Pre-Cambrian Faults and are not believed to involve faulting of the overlying Paleozoic units.

### **3.4 REGIONAL GROUND WATER CONDITIONS**

The near surfaces granular deposits and fill layers in the Detroit area typically contain groundwater, which is perched above the underlying clay strata. This groundwater forms an intermittent unconfined aquifer, which varies seasonably in depth and extent. In

addition, confined groundwater is often contained within relatively thin granular layers that are occasionally present within the thick cohesive deposits and / or hardpan present throughout the corridor areas. Such confined aquifers are usually limited in extent, and therefore, have limited recharge capabilities. However, surficial granular layers near the Detroit River shoreline can obtain hydraulic communication with the river, sometimes requiring extensive dewatering programs.

The higher elevation regions of north and west Wayne and Oakland Counties drive ground water levels within the bedrock along the study area on the United States side of the Detroit River. The bedrock is charged from these areas and the artesian pressure levels within the bedrock correspondingly decrease from north to south and west to east towards the river, which acts as a discharge for the area. Artesian pressure levels within and along the banks of the river are expected to be on the order of Elevation 176.7 m (EL 580 feet), which corresponds to 2 to 2.5 m (6 to 8 feet) above the river level. Portions of the overlying hardpan and granular soil contained within the hardpan can be expected to contain the same artesian conditions as the underlying bedrock.

### **3.5 REGIONAL SALT AND SOLUTION MINING ACTIVITIES**

The Michigan Basin is one of the largest areas of halite (salt-NaCl) deposition in the world. Salt has historically been mined from the Salina Formation (F, D, and B-Units) either directly in solid form as rock salt or as natural or artificial brine pumped from solution mining wells. Historically, salt was thought to have been removed near Crossing Corridor X-10 and X-11 from the F, D, and B-Units of the Salina Formation at depths of approximately 275 to 300m (910 to 980 feet), 380 to 395m (1,255 to 1,295 feet), and 435 to 500m (1,428 to 1,650 feet), respectively. The area beneath Detroit and Windsor within the Michigan Basin is currently mined using both solution mining techniques and conventional room and pillar excavation methods.

In general, solution mining consists of introducing water from the surface down a well casing between an outer casing and a central tube. The brine produced from the salt

dissolving in the water is recovered through the central tube. Cavities using this method are usually wider at the top of the stratum than at the bottom because the fresh water, which tends to stratify above the denser salt brine in the cavity, dissolves salt more rapidly near cavity roofs than at the base of the cavities, which are in contact with saturated brine. This would result in an inverted cone shaped cavity.

With continued production using this method, solution cavities often coalesce with adjacent cavities to form composite cavities called galleries. When this occurred historically, one or more of the wells were then converted to water inlet wells and the brine was pumped out through other wells in the interconnected system. As production continues in the gallery, large spans of unsupported roofs are sometimes created, which in turn could cause sagging, downward flexure, and local separation of rock units resulting in local roof collapse and eventual surface subsidence in some instances. Uncontrolled solution mining near the top of a salt layer commonly left overlying weak or weakened rock exposed at the top of the cavity, which increased potential for roof collapses. Historical reports and sources indicate that solution mining cavities beneath Zug Island, directly to the south and adjacent to the Crossing X-10 corridor, are interconnected. However, there has been no documented historical subsidence in this area.

The subsidence and/or collapse would progress upwards as a chimney effect on an acute angle from vertical from the outside edges of the cavity. Several theories have been published on the subsidence propagation to the surface, the more notable of which attributes surface daylighting to failure of the Sylvania Sandstone Formation at a depth of approximately 120 m (400 feet). According to the theory, the sandstone disintegrates under the induced compression from rock mass sagging, and the fragments filter downwards as granular material into voids below. This results in a void at a depth at approximately 120 m (400 feet) instead of the original cavity depth. This mechanism would explain why theoretical “bulking” of broken rock pieces would not be sufficient to fill the cavities before daylighting occurs.

Known and suspected areas of solution mining have been identified and discussed in NTH's Geotechnical Evaluation for the Proposed Detroit River International Crossing dated December 28, 2005 and NTH's draft Right of Entry Report, dated January 23, 2006. A comprehensive brine well investigation program is planned, and thusly, is not in the scope of this report.

#### **4.0 HISTORICAL GEOTECHNICAL AND ENVIRONMENTAL DATA**

The following historical data has been collected, to provide specific historical information for the proposed crossing corridors:

- Unpublished test boring data from the files of NTH Consultants, Ltd.
- Unpublished test boring data from the files of TolTest, Inc.
- Unpublished test boring data from the files of STS Consultants, Ltd.
- Unpublished Historical Geotechnical Data from the Detroit River Bridge Company
- Published well data from the Semet-Solvay Disposal Well Logs
- Detroit River Navigation Charts, Recreational Chart 14853 by The U.S. Department of Commerce 11<sup>th</sup> edition 1997.
- State of Michigan, Department of Natural Resources Geological Survey, Report on Investigation 3, Geology For Land and Ground-Water Development In Wayne County, Michigan, 1969 by Andrew J. Mozola.
- Geologic Atlas of the United States – Detroit Folio, by W. H. Sherzer, U.S. Geological Survey, 1916.

After reviewing this information, soil profiles were developed for both the X-10 and X-11 crossing corridors as shown on the attached Figure Nos. 2 and 3. These soil profiles are discussed in terms of geotechnical and environmental conditions as follows:

## **4.1 GEOTECHNICAL CONDITIONS**

### **4.1.1 Crossing X-10 Corridor**

In the X-10 Corridor, limestone bedrock (Dundee Limestone Formation) comprises the bedrock immediately below the soils at approximately Elevation 149 m (EL 489 feet) back from the Detroit River bank and decreases to approximately Elevation 148 m (EL 486 feet) at the river bank as shown on Figure No. 2. Bedrock is expected to dip beneath the river to a low point of approximately Elevation 146 m (EL 480 feet). Based on the historical data, the Dundee limestone in this area is higher permeability, typically in the range of  $10^{-2}$  to  $10^{-4}$  cm/sec, with the highest permeabilities near the soil rock interface.

Hardpan cover over the bedrock on the order of 2.5 to 3 m (8 to 10 feet) is expected. The bottom of the Detroit River within the navigation channel is expected to be approximately Elevation 164.5 m (EL 540 ft), resulting in soft ground cover on the order of 19 m (62 feet). Soft ground soils generally consist of soft to stiff silty clay away from the riverbank. At the river's edge, granular soils are expected with varying amounts of silt, clay, and gravel. Overlying the native granular soils, fill soils of varying type and consistency are expected, with the potential for environmental contamination and deleterious material.

### **4.1.2 Crossing X-11 Corridor**

At the X-11 Corridor, limestone bedrock (Dundee Limestone Formation) comprises the bedrock immediately below the soils with a surface generally expected to vary between approximately Elevation 146 m to 147.5 m (EL 480 to 484 feet) as shown on Figure 3. Bedrock is expected to rise beneath the river to a high point of approximately Elevation 151 m (EL 496 feet). Permeability is expected to be similar to X-10 as discussed above.

Hardpan cover over the bedrock on the order of 2.5 to 3 m (8 to 10 feet) is expected. The bottom of Detroit River is expected to be on the order of Elevation 163 m (EL 535 feet), resulting in soil cover of approximately 17 m (56 feet) over the bedrock. Soft ground soils generally consist of soft to stiff silty clay away from the riverbank. At the river's

edge, granular soils are expected with varying amounts of silt, clay, and gravel. Overlying the native granular soils, fill soils of varying type and consistency are expected, with the potential for environmental contamination and deleterious material.

## **4.2 SUMMARY OF ENVIRONMENTAL CONDITIONS**

Based on our experience along the Detroit River shoreline and within the Detroit River sediments, environmental issues will be present for any excavations along the United States shorelines and within the upper 2 to 3 meters (5 to 10 feet) of river sediment. Along the shoreline, fill soils to depths of 2 to 9 meters (5 to 30 feet) from previous activity are typically contaminated requiring disposal in Type II landfills. Within the river, sediments along the river bottom are also typically contaminated increasing in risk and contamination levels especially south of the downtown Detroit area.

### **4.2.1 Crossing X-10 (Former Solvay – Detroit Coke Site)**

The former Detroit Coke Site, originally owned by the Solvay Processing Company (Solvay), occupies most of the X-10 landing area between Jefferson Avenue and the Detroit River. The Detroit Coke Site was used for coke oven and coke oven gas by-products operations from early 1900 until 1991. Due to the presence of regulated deep underground injection wells in the western part of the property, it was identified as a Resource Conservation and Recovery Act (RCRA) facility. Associated environmental impacts with the coke oven and coke oven gas by-products operations included tar, free phase hydrocarbons (free product), and soil and groundwater contamination. Almost the entire site has been impacted by the former industrial operations.

Site soils are contaminated with VOCs, SVOCs, ammonia, cyanide, and metals at concentrations exceeding the MDEQ industrial criteria for indoor and ambient air, direct contact, particulate inhalation, and surface water protection. Site groundwater is contaminated with VOCs, SVOCs, ammonia, cyanide, and metals at concentrations exceeding the MDEQ industrial criteria for indoor air, direct contact, and surface water protection.

Honeywell, the current owner of the Detroit Coke Site and the primary responsible party, has installed a demarcation membrane in certain areas, and approximately 15 to 30 cm (6 to 12 inches) of clean fill material has been placed over the membrane to prevent contact with the impacted soil. However, this membrane and clean fill layer may not be present throughout the entire site. Honeywell has also installed groundwater collection trenches to limit impacted groundwater from discharging to the Rouge River and Detroit River.

The site may also have been used as a brine well processing facility, without any documented environmental impacts attributed to that operation.

#### **4.2.1.1 X-10 Disposal Wells**

Research for this project has also uncovered the existence of three previously operated deep disposal wells on the former Solvay/Honeywell (Crossing X-10) parcel. The wells were drilled from 1969 to 1978 to depths of greater than 1.2 km (4,000 feet). The wells were used to inject hazardous waste into permeable formations (Munising Group) deep within the ground. Wells #1 and #3 were plugged and abandoned according to the Michigan Department of Environmental Quality (MDEQ) and court proceedings, in which the operators of the hazardous waste injection operation were prosecuted for illegal activities. Well #2 is scheduled to be plugged during the winter of 2006, according to the MDEQ. Available lithology logs do not indicate the existence of solution voids encountered during drilling of any of these wells, one of which is apparently about 120 m (400 feet) from a documented Solvay brine well. From this information, it appears that at least in this location, the brine mining activities did not create voids more than 120 m (400 feet) from the actual brine well. It should also be noted however, that the logs of the injection wells do not contain great detail, and may not have documented small voids encountered.

#### **4.2.2 Crossing X-11 (Former Revere Copper and Brass)**

The former Revere Copper and Brass site occupies the southern portion of the X-11 Crossing between Jefferson Avenue and the Detroit River and was used for manufacturing copper and brass products from the early 1900's until 1985. In addition, significant portions of the site were filled with debris resulting from land reclamation on the site. Contamination generally consisting of volatile organic compounds (VOCs), semi-volatile organic compounds (SVOCs), metals and polychlorinated biphenyls (PCBs) remains at the site in excess of Michigan Department of Environmental Quality (MDEQ) Part 201 Residential and, Commercial and Industrial criteria. The Mistersky Power Plant occupies the middle to northern region, and although not documented, environmental concerns may persist here as well.

### **5.0 CONCEPTUAL CROSSING STRUCTURE REQUIREMENTS**

The project team has developed the crossing concept as a three-lane each way bridge crossing. For a three-lane crossing, a roadway width on the order of 30 m (100 feet) will be required.

Given the navigational requirements, the bridge is anticipated to be a suspension bridge or cable-stayed bridge with primary piers on or near the shoreline, with secondary supports back from the shoreline. For the purposes of this document, primary foundation elements are defined as the main structural foundation (for cable stay and suspension bridges) and anchorage piers for the suspension bridge. Secondary foundation elements are defined as foundation elements for the approach roadway piers to the bridge and will not be discussed further in this report. Primary anchor piers would be located 300 to 450 m (1,000 to 1,500 feet) behind the primary piers, which would be located at or near the river's edge. Foundations for primary bridge piers would be on bedrock. These foundations could be constructed as large diameter sinking shafts or drilled caissons, or driven or drilled piles with a pile cap.

## **6.0 EVALUATIONS, ANALYSIS, AND RECOMMENDATIONS**

### **6.1 SOIL CONDITIONS**

Based on the results of our historical investigation, the existing fill deposits at both crossing locations are highly variable and are not considered suitable for support of any foundation elements. However, provided that earthwork operations are followed as described later in this report, and that some settlement can be tolerated, the fill deposits may be acceptable for pavement, sidewalks, etc.

The underlying desiccated silty clay and granular soils are considered suitable for support of moderate foundation loading such as support buildings, but not the heavy loading from primary or secondary bridge foundation elements.

The hardpan soils underlying both corridors are considered well suited for the heavy foundation loading anticipated from proposed secondary structural elements of the bridge.

The bedrock underlying the hardpan soils is considered well suited for the heavy foundation loading anticipated from primary and secondary foundation elements of the bridge.

### **6.2 GROUNDWATER AND GAS CONSIDERATIONS**

Due to the depth of the proposed excavations, groundwater control will be required to address groundwater conditions within the granular soil portions of the soft ground profile above the hardpan, as well as artesian conditions within the hardpan, possible granular soils within the hardpan, and within the bedrock. The groundwater within the hardpan and bedrock typically contains dissolved sulfides, which can create hydrogen sulfide gas upon exposure to the atmosphere and groundwater discharge concerns if not addressed. Likewise, toxic or explosive gases may be potentially present in localized areas throughout the site.

Given the anticipated relatively high rock permeabilities, it appears groundwater in the bedrock will require control as part of the caisson construction. This is typically accomplished in the area by either rock grouting, or by tremie placement of the concrete for the caissons. Without specific data for the foundations, and considering the importance of the primary foundation elements, we cannot at this time assess the feasibility or practicality of tremie caissons.

Rock grouting of the Dundee limestone would allow for exposing and cleaning the rock surface, to confirm the primary and secondary foundation elements are founded on an adequate bearing surface. However, rock grouting of the upper weathered rock will require considerations for granular soils if present immediately over the bedrock. The granular soils have sometimes been observed to fill the upper rock fractures on other projects, which may decrease the ability of the grout from the grouting program to penetrate into the upper bedrock fractures. If this were found to be the case, groundwater control at the soil/rock interface could be accomplished by dewatering and or a soil stabilization program. Any groundwater produced from pumping is expected to require treatment for dissolved sulfides and hydrogen sulfide prior to disposal. Additionally, substantial odor control will be required for airborne hydrogen sulfide gas.

## **6.3 FOUNDATION RECOMMENDATIONS**

### **6.3.1 Primary Foundation Elements**

As discussed above, we expect a deep foundation system will be required to support the proposed primary bridge elements. Based on the overall evaluation of the historical subsurface data developed in this investigation and consideration of the project background information, we recommend at this time that the deep foundation system be planned to consist of straight-shaft drilled concrete filled caissons bearing in competent bedrock. The caissons should extended through the upper fill, silty clay, granular soil layers, hardpan soils, and be founded at least 5 feet into the underlying limestone bedrock formation, resulting in depths of approximately 35 m (115 feet). This will minimize uncertainties in the design by providing a uniform and reliable bottom pier elevation

bearing on competent rock. As indicated in our Review of Allowable Bearing Capacity for Drilled Shafts on Rock Memorandum, dated April 18, 2006, a net allowable bearing pressure of 12.87 MPa (120 tons per square foot) may be used for conceptual design purposes.

We understand that a preliminary shaft diameter of approximately 305 cm (120 inches) is necessary from a foundation load standpoint. In any case, for planning purposes, caissons should be spaced a minimum of one diameter apart (edge to edge). In addition, during the conceptual design of foundation systems expected to be subjected to lateral loading, preliminary values for the modulus of lateral subgrade reaction can be applied as follows:

<u>Depth (ft)</u>	<u>Lateral Subgrade Reaction Modulus- <math>\text{kN/cm}^3</math> (kci)</u>
Granular Layers	16.3 (60)
Cohesive Layers	27.1 (100)
Hardpan	48 (176)
Bedrock	80 (295)

If used in the structural analyses, the above moduli should be used in conjunction with caisson diameters, modeling method used, etc., to determine appropriate lateral caisson capacities. Special care should be taken to ensure proper units are maintained.

Reviews of several of the historical test borings extending to bedrock reveal granular layers extending to the bedrock at locations approaching the shore of the Detroit River. We expect that the predominately sandy layer(s) will generally possess little to no standup time. In locations with this condition, it will be necessary to extend steel casing entirely to the caisson base due to collapsing sands.

In order to evaluate the feasibility of the drilled excavations in the soft to medium clay soil zones, probable overload factors, which are a ratio of overburden stress to soil shear

strength, were calculated. Overload factors on the order of six to eight (6 to 8) typically indicate marginal sidewall stability, and values greater than eight (8) typically indicate squeezing conditions. Based on the historical soil data, it is estimated that the overload factors will approach twenty (20) as the excavation depth below the ground surface reaches the hardpan layers. This indicates that squeezing conditions will be present within the shaft excavations. As such, we expect that casing will be required for the full depth of the caissons through clay as well as sand.

Based on our experience with subsurface conditions in the Detroit Area, as well as observations from test boring programs on nearby parcels, the possibility of random occurrence of toxic, noxious, and explosive gases in caisson excavations cannot be precluded, although proper gas monitoring will minimize the risk associated with such events.

#### **6.4 GENERAL FOUNDATION COMMENTS**

All proposed foundation locations should be further investigated with a comprehensive geotechnical investigation once a final crossing alignment and main foundation element locations have been determined.

#### **6.5 SEISMIC DESIGN CRITERIA**

The 2003 Michigan Building Code (MBC) states that the site shall be classified as one of the site classes defined in Table 1615.1.1. Where the soil shear velocity is not known, site class shall be determined, as permitted in Table 1615.1.1, from standard penetration resistance or from soil undrained shear strength, calculated in accordance with Section 1615.1.5. We anticipate the site soil profile does not contain any soils having one or more of the characteristics that would require the site to be classified as Site Class F. Therefore, according to Section 1615.1.5.1 of the MBC, we anticipate the site shall be classified as Site Class E. A site-specific investigation including determination of shear wave velocities for the various soil layers should be performed prior to actual design of foundation elements.

## 7.0 LIMITATIONS

The evaluations and preliminary recommendations presented in this report have been formulated on the basis of generalized data in the vicinity of the proposed bridge crossings, together with current preliminary concepts for the bridge and foundations. As such, all of the preliminary conclusions presented herein are considered appropriate for concept-level evaluations of the design, and for concept-level cost estimating. This report is not considered appropriate for use in preliminary or final design of the structure. Experience indicates that the actual sub-soil conditions at the corridors will vary from those generalized on the basis of the historical information derived for this report. On this basis, a comprehensive site-specific geotechnical investigation should be performed prior to the design of any foundations systems for the proposed structure. This may be staged as a “Phase 1” investigation that would provide some information for a preliminary design, followed by a comprehensive “Phase 2” investigation that would provide geotechnical exploration and analysis at the locations of each primary and secondary foundation element.

The scope of the present investigation was limited to the concept-level evaluation of subsurface conditions for the support of the proposed bridge foundations. Considerations relating to environmental concerns beyond those specifically mentioned in this text, or other possible regulatory restrictions on development were not included in the scope of this investigation.

Respectfully Submitted,  
NTH Consultants, Ltd.

Craig R. Johnson  
Project Engineer

Joseph B. Alberts, P.E.  
Asst. Project Manager

Fritz J. Klingler, P.E.  
Project Manager

CRJ/JBA/FJK  
Attachments



**LEGEND**

-  APPROXIMATE CROSS SECTION LOCATIONS
-  SOLVAY DISPOSAL WELLS



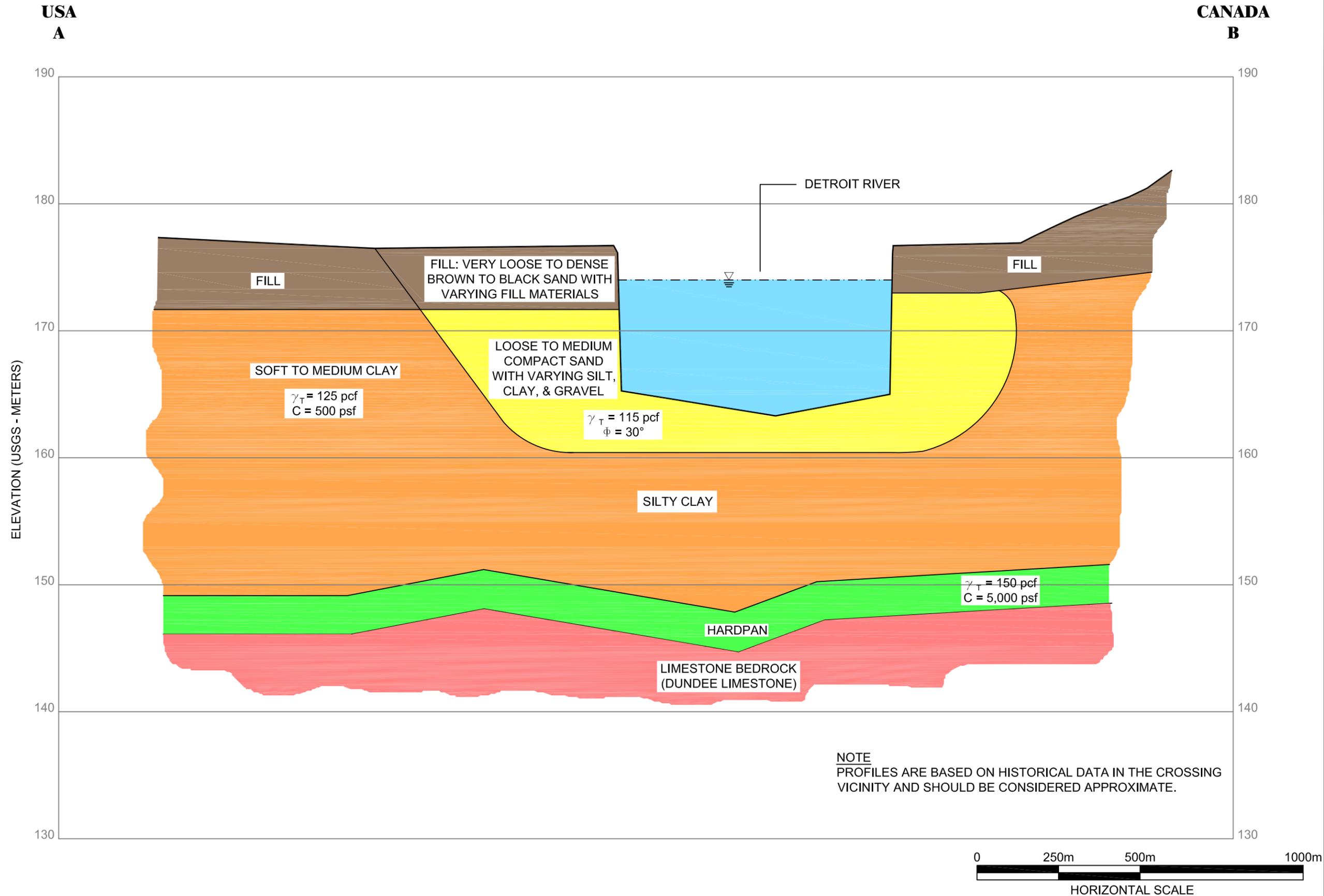
**NTH Consultants, Ltd.**  
Infrastructure Engineering  
and Environmental Services



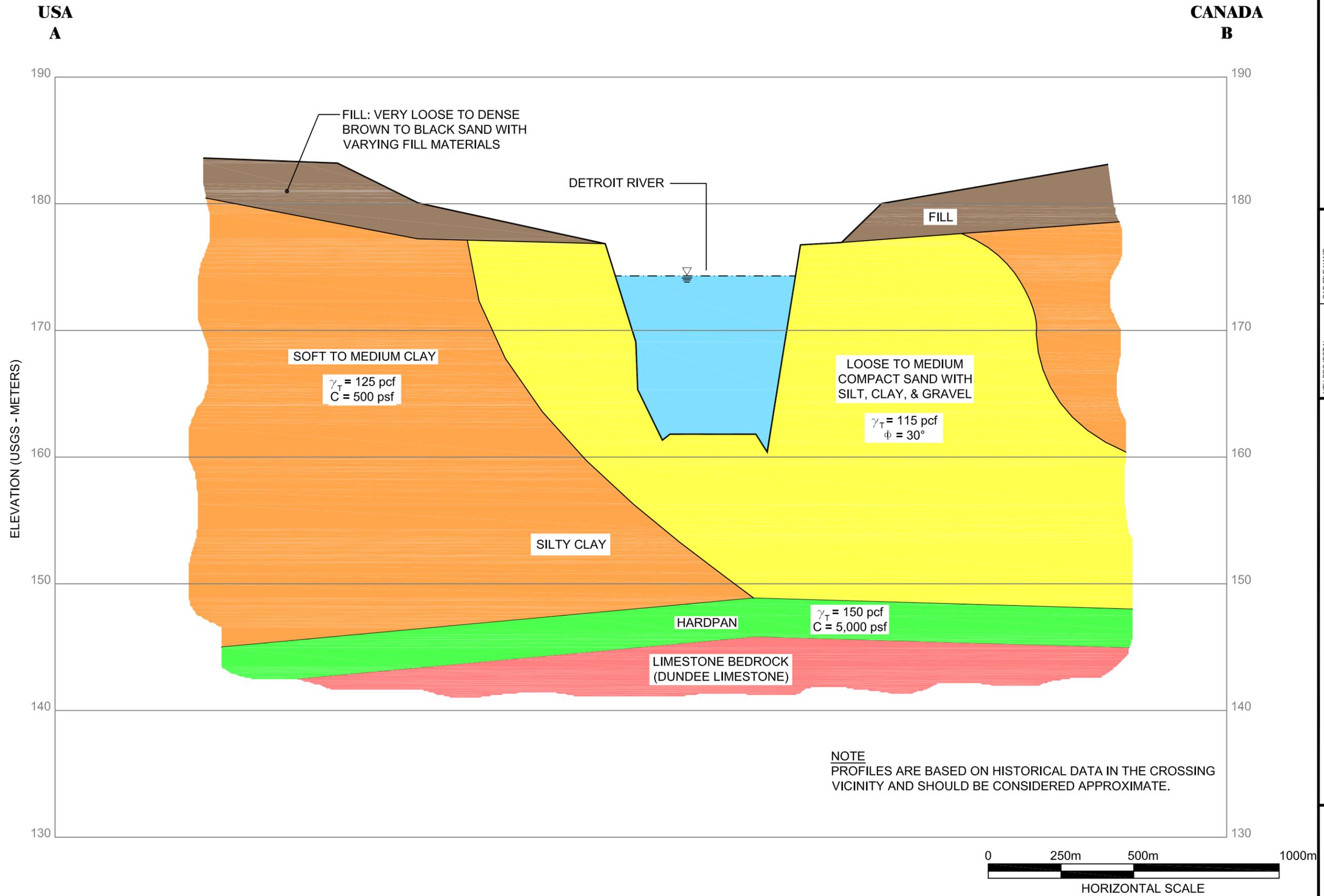
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DESIGNED BY: CRJ	INCEP DATE: 05 SEP 2006
DRAWN BY: SHB	DRAWING SCALE: AS SHOWN
CHECKED BY: JBA	PLOT DATE: 05 SEP 2006

**PRELIMINARY GEOTECHNICAL EVALUATION**  
**DETROIT RIVER INTERNATIONAL CROSSING**  
**DETROIT, MI - WINDSOR, ON**

FIGURE No.  
**1**



<b>NTH Consultants, Ltd.</b> Infrastructure Engineering and Environmental Services			
<small>CAD FILE NAME:</small> D05001432	<small>INCEP DATE:</small> 05 SEP 2006	<small>DRAWING SCALE:</small> AS SHOWN	<small>PLOT DATE:</small> 20 SEP 2006
<small>NTH PROJECT No.:</small> 15-050014-01	<small>DESIGNED BY:</small> CRJ	<small>DRAWN BY:</small> SHB	<small>CHECKED BY:</small> JBA
<b>CROSS SECTION AT X10 CORRIDOR</b>			
<b>DETROIT RIVER INTERNATIONAL CROSSING</b>			
<b>DETROIT, MICHIGAN - WINDSOR, ONTARIO</b>			
<small>FIGURE No.</small>			
<b>2</b>			



<b>NTH Consultants, Ltd.</b> Infrastructure Engineering and Environmental Services	
<b>CAD FILE NAME:</b> D05001432	<b>INCEP DATE:</b> 05 SEP 2006
<b>NTH PROJECT No.:</b> 15-050014-00	<b>DESIGNED BY:</b> CRJ
<b>DRAWN BY:</b> SHB	<b>DRAWING SCALE:</b> AS SHOWN
<b>CHECKED BY:</b> JBA	<b>PLOT DATE:</b> 20 SEP 2006
<b>CROSS SECTION AT X11 CORRIDOR</b>	
<b>DETROIT RIVER INTERNATIONAL CROSSING</b>	
<b>DETROIT, MICHIGAN - WINDSOR, ONTARIO</b>	
<b>FIGURE No.</b>	
<b>3</b>	



thin layer of dense glacial till overlies the bedrock.

## **Sedimentary Geology**

The study area is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plains. Within this region, Essex County and the southwestern part of Kent County are normally discussed as a subregion known as the Essex Clay Plain. The clay plain was deposited during the retreat of the ice sheets (late Pleistocene Era) when a series of glacial lakes inundated the area. In general, the ice sheets deposited till in the area of Windsor and Detroit. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the till may have been directly deposited at the contact between the ice sheet and the bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of the ice may have been deposited through the lake water (lacustrine). Glacial till, in its common usage, often indicates a very dense or hard composition resulting from consolidation and densification under the weight of the ice sheet. The mineral soil particles typically have a distribution of grain sizes ranging from cobbles to clay. However, in many areas of Windsor and Detroit, the soils described as “glacial till” were deposited through water and have a softer consistency as a result.

The major clay stratum, typically ranging in thickness from about 30 m to 35 m in the X10 and X11 crossing areas, exhibits a till-like structure exemplified by a random distribution of coarser particles within the primarily fine-grained silt and clay deposit (this type of deposit is also called “diamict”). The near-surface clay is generally stiff to hard and brown and exhibits undrained shear strengths in the range of 100 to 200 kPa or more. Underlying this stiff to hard “crust” the silty clay becomes grey-brown, firm to stiff, and exhibits undrained shear strengths in range of 60 to over 150 kPa. In the vicinity of the potential X10 and X11 bridge crossing areas below the groundwater level, the undrained shear strength of the silty clay can be as low as 10 to 20 kPa, but is more typically in the range of 20 to 40 kPa based on recent explorations at the intersection of Ojibway Parkway and E.C. Row Expressway (see attached geotechnical summary of borehole BH/FV/CPT-23, Figure 1).

## **Bedrock Geology**

Within the Windsor area, the bedrock geology consists of an evaporate-carbonate sequence of rock formations. These include the Silurian Salina formation, the Devonian Bass Islands dolomite, the Detroit River Group, the Dundee Formation, and the Hamilton Group, respectively, with decreasing age and closer proximity to the ground or bedrock surface. The surface of the bedrock, beneath the overlying sediments, is relatively flat except for “a significant depression in the vicinity of the Windsor airport. The depression may represent a dissolution collapse of either the underlying carbonates or the lower Salina salt beds” (Hudec 1998).

Devonian Age bedrock of dolomite, shaly limestone, limestone and sandstone extend from the bedrock surface, found at depths of between 30 and 35 m, to depths of about 160 m below ground level. Unconfined compression strength of this first bedrock formation (Detroit River Group, Lucas Formation) has been reported to be between about 50 and 100 MPa for fresh, sound bedrock specimens. These bedrock formations are underlain by the Salina Group of formations that include thick salt beds at depths of about 270, 300, and 400 m below the ground surface. It is also known that relatively small volumes of petroleum are found within the limestone and dolomite strata.

## **Hydrogeology**

Static groundwater levels within the overburden soil deposits are typically at about 1 and 3 m below the ground surface depending on specific locations and ground surface elevations. Groundwater within the underlying glacial till and bedrock in some areas, however, is known to be under artesian pressures (in which groundwater levels will rise above the ground surface for wells that penetrate the soil overburden and connect with groundwater in the bedrock). In these areas, particularly in the western part of the study areas, artesian pressures may be on the order of 2 to 3 m above the river level. In general, groundwater flow will be toward the Detroit River, Lake St. Clair, and Lake Erie. Groundwater from within the bedrock is likely to be corrosive because of the salt deposits found at depth. Recent explorations and experience drilling on the potential X10 North and X11 crossing sites have encountered artesian water conditions with estimated static pressure head levels of about 1.5 m above existing ground levels.

## **Gas**

It is also known in some areas that the groundwater contains hydrogen sulphide that will be liberated from solution and become hydrogen sulphide gas at normal atmospheric pressures. Hydrogen sulphide gas is toxic at low concentrations. Hydrogen sulphide was encountered during drilling on the potential X10 North and X11 crossing sites as well as during drilling for the approach corridor near the intersection of Ojibway Parkway and E.C. Row Expressway, though levels were not sufficient to result in health and safety concerns or odour problems. Methane gas has also historically been encountered during excavations into both soft ground and bedrock in the Detroit-Windsor area. Methane gas can present an explosion hazard if not adequately controlled during construction. Methane gas, however, has not been encountered during drilling on the potential crossing sites up to the date of this memorandum.

## **Structure Foundations**

In some areas, it may be feasible to support relatively lightly loaded structures on shallow spread foundations seated on the surficial sand deposits or the stiffer parts of the silty clay deposits. However, the feasibility of this foundation option will be highly dependent upon local soil conditions, foundation loads, and performance (settlement) requirements. It is understood that

some portions of the approach structures for the Ambassador Bridge are supported on shallow foundations. In the area of the potential X10 and X11 crossings, however, it is anticipated that the use of shallow spread foundations will be limited and that the majority of structures may require deep foundations.

For preliminary route option planning, it should be assumed that any moderately to highly loaded structures (buildings and bridges) will need to be supported by deep foundations bearing on the glacial till or bedrock. In the Windsor area, structure foundations often consist of driven steel H-piles. It is likely that such driven pile foundations may be required for highway overpass structures constructed along the potential routes joining Highway 401 with the crossing location. Ultimate limit states capacities for typical HP310x110 end-bearing piles in the Windsor area are of the order of about 2,000 kN per pile, depending on the end bearing stratum and the spacing of piles within pile groups. If down-drag loads are induced by embankment or other fills constructed over soft soils, these will have the effect of reducing the ultimate capacity available to support structures. Although this end-bearing capacity value may be used for feasibility and preliminary design evaluations, it must be considered approximate and should not be used for any final design. Final design capacities for both ultimate and serviceability limit states must be based on site-specific explorations and analyses. The subsurface conditions within the potential corridor areas are such that friction piles may not be suitable and are not given further consideration in this report.

Drilled shaft foundations may also be used for support of heavily loaded structures. Construction of drilled shaft foundations may be complicated by the presence of artesian groundwater pressures, methane, or hydrogen sulphide gases which are largely dependent on the depth of drilling into bedrock, groundwater inflows, local artesian pressures, and gas concentrations. Typical end bearing capacities on the order of 6 MPa may be assumed for design for drilled shafts bearing on sound limestone bedrock, depending on local bedrock quality and weathering, the spacing of drilled shafts within any groups of drilled shafts, the potential presence of discontinuities or vugs (small voids within the rock mass), and tolerable displacements may influence the choice of final design bearing pressures. Although these end-bearing capacity values may be used for feasibility and preliminary design evaluations, it must be considered approximate and should not be used for any final design. When higher shaft capacities are required, the shafts can be “socketed” into rock, with the capacity based on the adhesion developed between the shaft concrete and rock wall of the socket or based on the composite action of shaft adherence and end bearing. Final design capacities must be based on site-specific explorations and analyses.

Heavily loaded bridge foundations, of the type that may be needed for large-span structures crossing the Detroit River, have often been constructed using deep “caissons”. Although this term is often locally applied to drilled shaft foundations, bridge caissons usually consist of relatively large structures built by:

- constructing a perimeter form, either circular or rectangular, at the ground surface using timbers, steel, or concrete (precast or cast-in-place) that encompasses the final foundation plan shape;
- excavation is then carried out within and immediately beneath the edges of this form and the form is permitted to “sink” to the bottom of the excavation – in some cases, the edge of the form is created to act as a cutting edge;
- the height of the perimeter form is then built up, and the excavation sequence is carried out once again;
- this process is repeated until the final excavation depth and bearing stratum is reached (thus building the support for the excavation as the excavation proceeds); and
- the excavated interior of the form is filled with mass concrete creating a large foundation column to support the superstructure.

This method has been used to construct many of the foundations for major bridge crossings around the world, likely including the Ambassador Bridge foundations built in the 1920s. Often, to counteract groundwater pressures or the tendency of soft soils to squeeze into the caisson at its base during construction, excavation within the caisson is completed under compressed air. Because of health and safety concerns, recent work of this type has also been conducted using slurries, with all of the excavation and concrete placement work conducted under water. Similar to drilled shaft foundations, construction of drilled shaft foundations may be complicated by the presence of artesian groundwater pressures, methane, or hydrogen sulphide gases. As with other foundation types discussed above, final design capacities must be based on site-specific explorations and analyses.

## **BACKGROUND SEISMIC ANALYSIS AND DESIGN METHODS**

### **Canadian Highway Bridge Design Code**

The 2000 Canadian Highway Bridge Design Code (CHBDC) is the fundamental specification for bridge design in Canada and it is based on seismic hazard as defined in the 1995 National Building Code of Canada (NBCC). The seismic hazard is defined using the Zonal Acceleration Ratio (A), defined as in NBCC 1995. The design earthquake is defined as having a probability of exceedance of 10% in 50 years. In the 2000 CHBDC Bridge design specifications an Elastic Seismic Response Coefficient (C<sub>sm</sub>) is used to define the spectral shape. The spectral shape is a function of the Acceleration Coefficient (A), given in the CHBDC, the site coefficient, the importance factor and the period of the bridge. The soil profile types and site coefficients are similar to those in NBCC 1995. The Importance factor (I) is used to scale the elastic seismic

response coefficient. For lifeline bridges  $I=3.0$ , for emergency-route bridges  $I=1.5$  and for other bridges  $I=1.0$ .

Using this methodology the soil profile type (based on BH/FV/CPT-23) would be type 3, soft to medium stiff clays and sands and the site coefficient,  $S$ , would be 1.5. The zonal acceleration ratio for Windsor is 0, but a minimum value of 0.05 is used to construct the acceleration spectra as per the CHBDC. Figure 2 (attached) shows the acceleration spectra for Windsor with an importance factor of 1.0. It should be noted that the structures associated with the Detroit River International crossing may be considered lifeline or emergency route bridges in which case the design spectra would be multiplied by the corresponding importance factor.

### **National Building Code of Canada 2005**

The National Building Code of Canada was published in 2005 with an updated seismic analysis and design methodology. Seismic hazard is now defined by uniform hazard spectra (UHS) at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s. The probability of exceedance of the seismic hazard specified by means of the UHS is 2% in 50 years. In the 2005 edition of NBCC, the 1994 National Earthquake Hazard Reduction Program (NEHRP) site categories and response factor are adapted to the reference ground condition for Canada. The reference ground condition adopted by the 2005 NBCC is Site Class C. The 2005 NBCC method defines the site class by the shear wave velocity, undrained shear strength or standard penetration resistance in the top 30 meters of soil. There are 6 site classes from A to F, decreasing in soil strength from A, hard rock to E, soft soil, with site class F, to denote particularly vulnerable soils. The site class is determined to obtain soil factors,  $F_a$  and  $F_v$  used to modify the UHS to account for the affects of soil conditions in design. The 2005 NBCC uses an importance factor  $I_e$  to multiply the base shear for seismic design. Normal structures are assigned an  $I_e = 1.0$ , high importance category structures are assigned  $I_e = 1.3$  and Post-Disaster structures are assigned an  $I_e = 1.5$ . It should be noted that the importance factor is not applied directly to the spectral acceleration used in seismic design.

Using the NBCC 2005 methodology the soil profile type (based on BH-23) would be type site class E, soft soil with an undrained shear strength less than 50 kPa. The  $F_a$  and  $F_v$  values would be 2.1. The reference spectral acceleration coordinates for Windsor are  $S_a(0.2)=0.18$ ,  $S_a(0.5)=0.086$ ,  $S_a(1.0)=0.04$ ,  $S_a(2.0)=0.011$  and  $PGA=0.12$ . Figure 2 shows the spectral acceleration for Windsor site class E, with an importance factor of 1.0 applied.

### **ATC 2003 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges**

In 2003 ATC/MCEER published seismic design guidelines to be used as a supplement to the AASHTO bridge design specifications. It is anticipated that these new guidelines would form the basis for the next revision to both AASHTO and CHBDC seismic design codes. The changes in ATC/MCEER 2003 guidelines include the adoption of new USGS maps, more clear performance

objectives, design incentives, new soil factors and new spectral shapes. In 1996 the USGS published new seismic hazard maps to be used in the United States (Frankel et. al., 1996). The new seismic hazard values are presented as contour maps or tabulated values. The PGA values and the spectral acceleration at 0.2, 0.3 and 1 seconds are given. The USGS has presented these values at three probability levels: 10% in 50 years; 5% in 50 years; and 2% in 50 years. The spectral shape recommended in the 2003 guidelines is based on the 0.2 second and 1 second spectral accelerations. The values of the 0.2 second and 1 second spectral acceleration are determined based on the uniform hazard spectra procedure and as such, both values have the same probability of exceedance. The spectral shape used in the 2003 guidelines increases in the short period range to a plateau level and then decreases in the long period range. The 2003 ATC/MCEER guidelines also adopt the site class and site factors recommended by NEHRP in 1994 (also adopted in 2005 NBCC). The reference ground condition considered in the 2003 ATC/MCEER guidelines is site Class B, rock. Whereas in the 2005 NBCC the reference ground condition is site class C, soft rock. Therefore the  $F_a$  and  $F_v$  values used in design by the procedures of the two codes are slightly different. The 2003 guidelines for the Seismic Design of Highway Bridges uses Seismic Performance objectives based on Life Safety or Operational criteria, no importance factors are used.

Using the ATC 2003 methodology the soil profile type (based on BH/FV/CPT-23) would be type site class E. The  $F_a$  and  $F_v$  value would be 2.5 and 3.5 respectively. The reference spectral acceleration coordinates are  $S_a(0.2)=0.12$ ,  $S_a(1.0)=0.04$  and  $PGA=0.06$  for a probability of exceedance of 2% in 50 years for Detroit. (Note: the USGS and GSC have not developed a consistent framework for hazard definition, thus the GSC defines the PGA for 2% in 50 years as 0.12). Figure 2 shows the spectral acceleration for site class E.

## **SEISMIC HAZARD ASSESSMENT**

The site location has historically been considered to be in an area of low seismicity with PGA values of less than 0.05g from an earthquake with a 10% probability of exceedance in 50 years. New hazard models and a move to design earthquakes with a 2% probability of exceedance in 50 years now define reference (Site Class C) PGA values in the order of 0.12g.

To reflect the actual Site Class condition of E, the site specific PGA value would be amplified to about 0.25g which represents a moderate level of ground shaking. Such ground shaking could be reflected in a potential for seismic liquefaction in loose, saturated granular deposits. However, the borehole data does not indicate the presence of such deposits at the site, though site specific investigations must be completed prior to final design as it is understood that such granular deposits may be found in some locations adjacent to the Detroit River.

Nonetheless, the seismic stability of earthen embankments and the shoreline should be assessed in consideration of the moderate level of ground shaking. In addition, retaining walls will need to consider the lateral pressures induced by such seismic shaking.

The proposed long span cable stayed bridge structure over the Detroit River would be sensitive to seismic excitation. Although the specific bridge location is not yet defined, the site soil conditions of the bridge are likely to be similar to those found in BH/FV/CPT-23. Therefore a site class of E would be appropriate for the bridge site. A cable stayed bridge structure with a span length of 500 to 900 metres would likely have a fundamental period of greater than 1.0 seconds. As can be seen from Figure 2, for period ranges greater than 1.0 seconds the differences between the design spectral values of the NBCC 2005, ATC 2003 and CHBDC 2000 are relatively small (when an importance value of 1.0 is used). However, such a structure (an international crossing between Canada and the United States) may be considered a lifeline structure according to the CHBDC, for which the design spectra (and any differences between spectra) would be multiplied by a factor of 3. According to the ATC 2003 guidelines the performance objective which may be appropriate for this structure could be the Operational performance level, in which case the level of service required following the maximum considered earthquake (2 % in 50 years) would be immediate and the amount of damage to the structure would be expected to be minimal. If the bridge is considered to fall into these design categories, a more stringent and rigorous seismic analysis and design would likely be required.

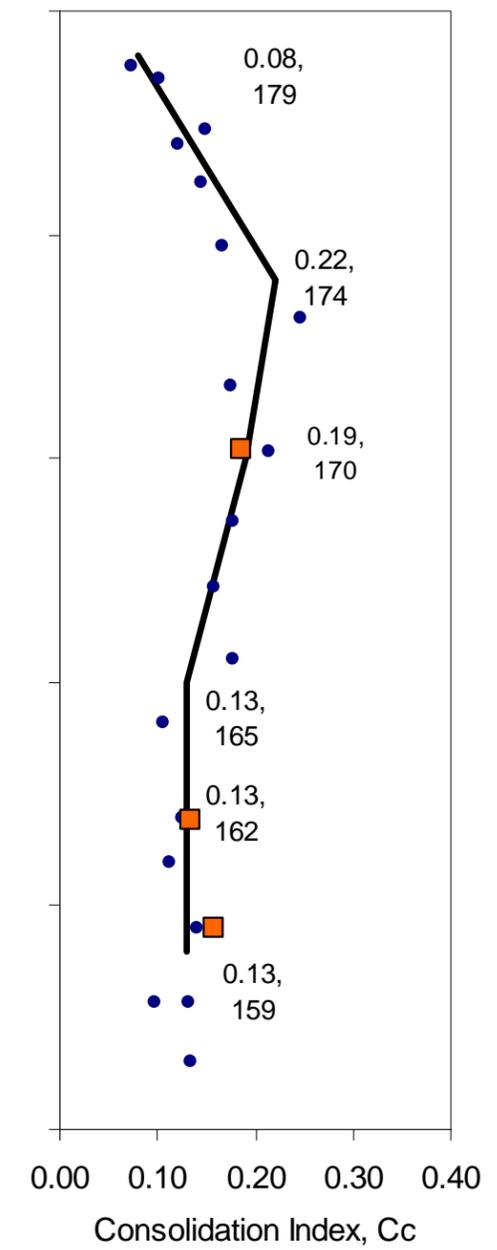
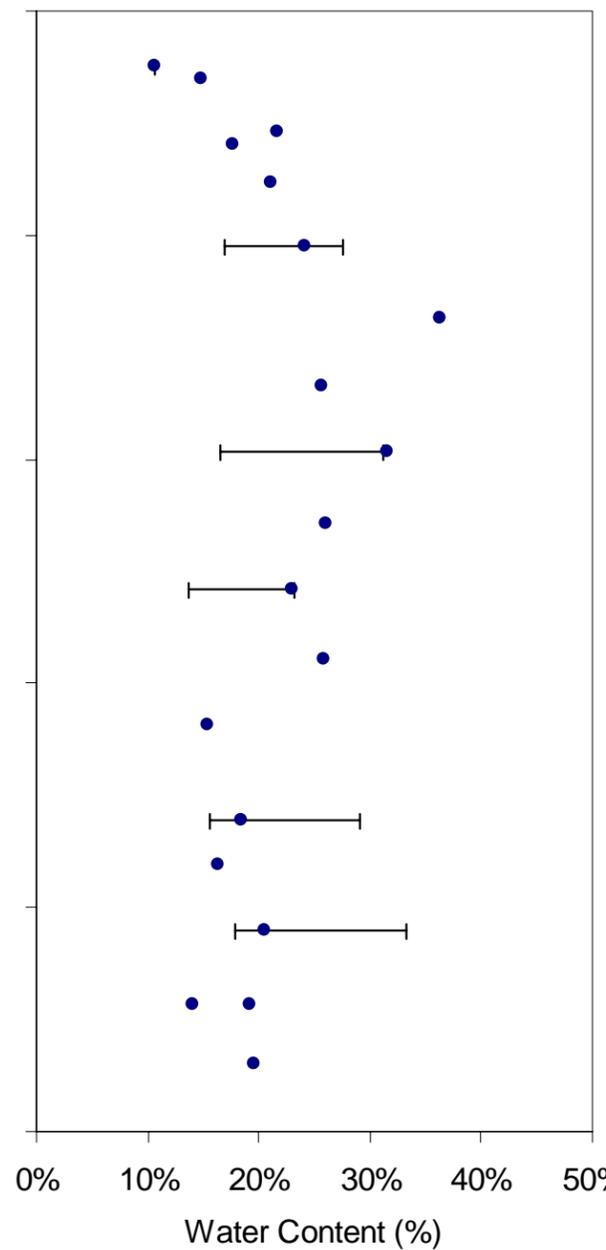
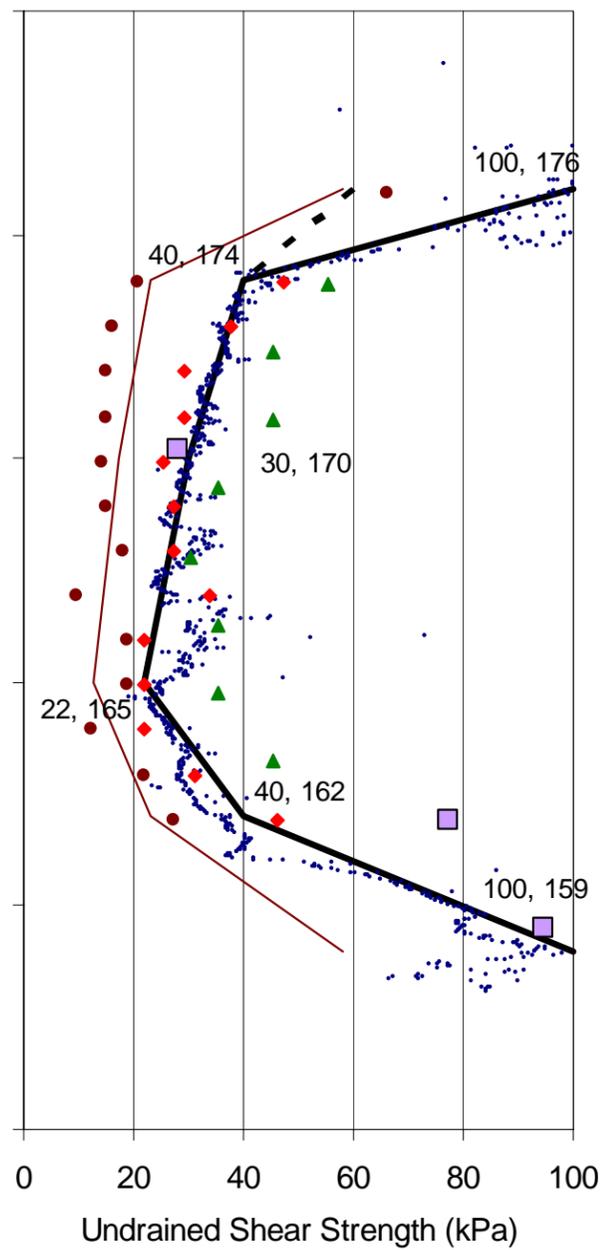
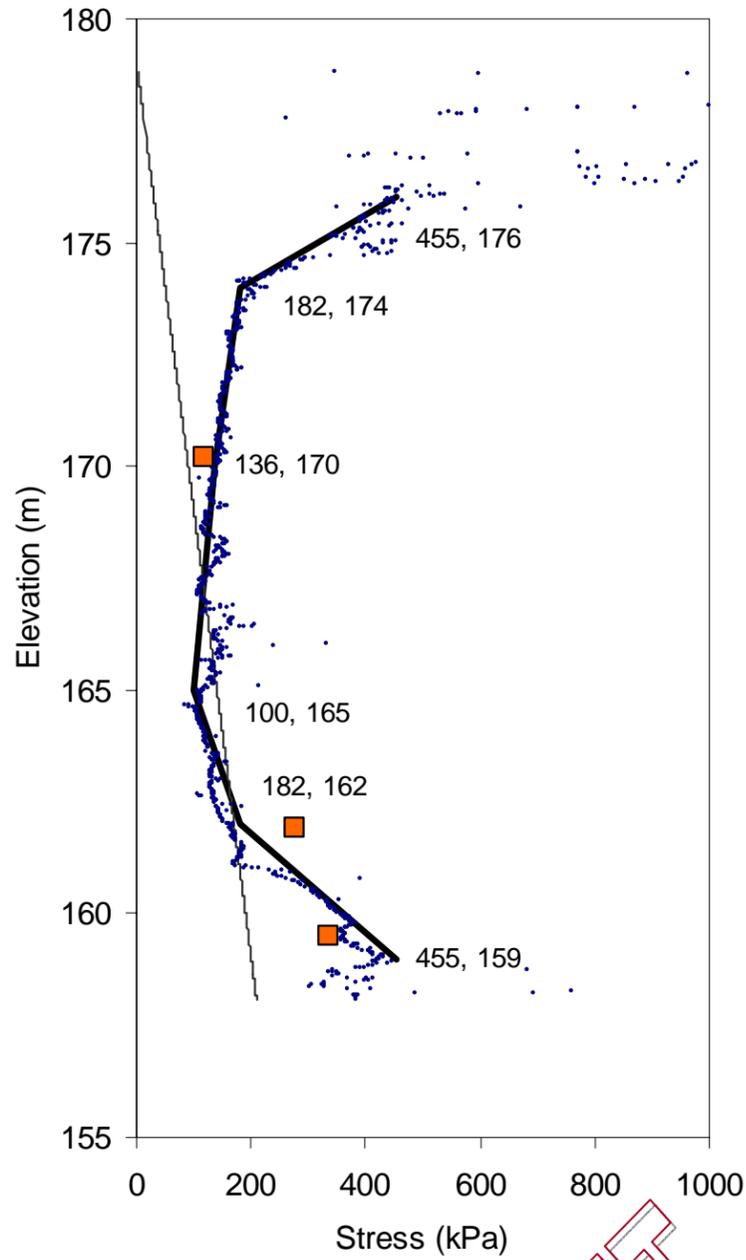
### **Seismic Design Conclusions**

Because of recent developments in the quantification of seismic hazard the CHBDC 2000 will likely be updated to adopt the specification of seismic hazard in terms of the UHS at 2% in 50 years (Adams et. al., 2003) and the NEHRP 1994 site classification system. However in order to incorporate the new information and practices, the CHBDC 2000 method of seismic analysis and design need to be modified, much as NBCC had to update their seismic methodology from 1995 to 2005. The recommended LRFD guidelines published in 2003 by ATC/MCEER provide a likely framework to incorporate these changes into the seismic analysis and design methodology of the next generation CHBDC.

It is suggested that a design approach based on performance based seismic design using the ATC 2003 performance objectives and the NBCC 2005 seismic hazard definition and site factors be used for the Detroit River International Crossing Project.

/

document4



— Design Line  
 — Effective Stress  
 • Preconsolidation Pressure (CPT)  
 ■ Oedometer

— Design Line  
 • Su (CPT)  
 ◆ Su, Natural (Nilcon)  
 ▲ Su, Natural (B & N Vanes)  
 ■ Su (CIUC)  
 ● Post-Peak Su (Nilcon)  
 — Post-Peak Design  
 - - - Su (Crust)

Plastic Limit    Liquid Limit  
 ————  
 • Natural Water Content

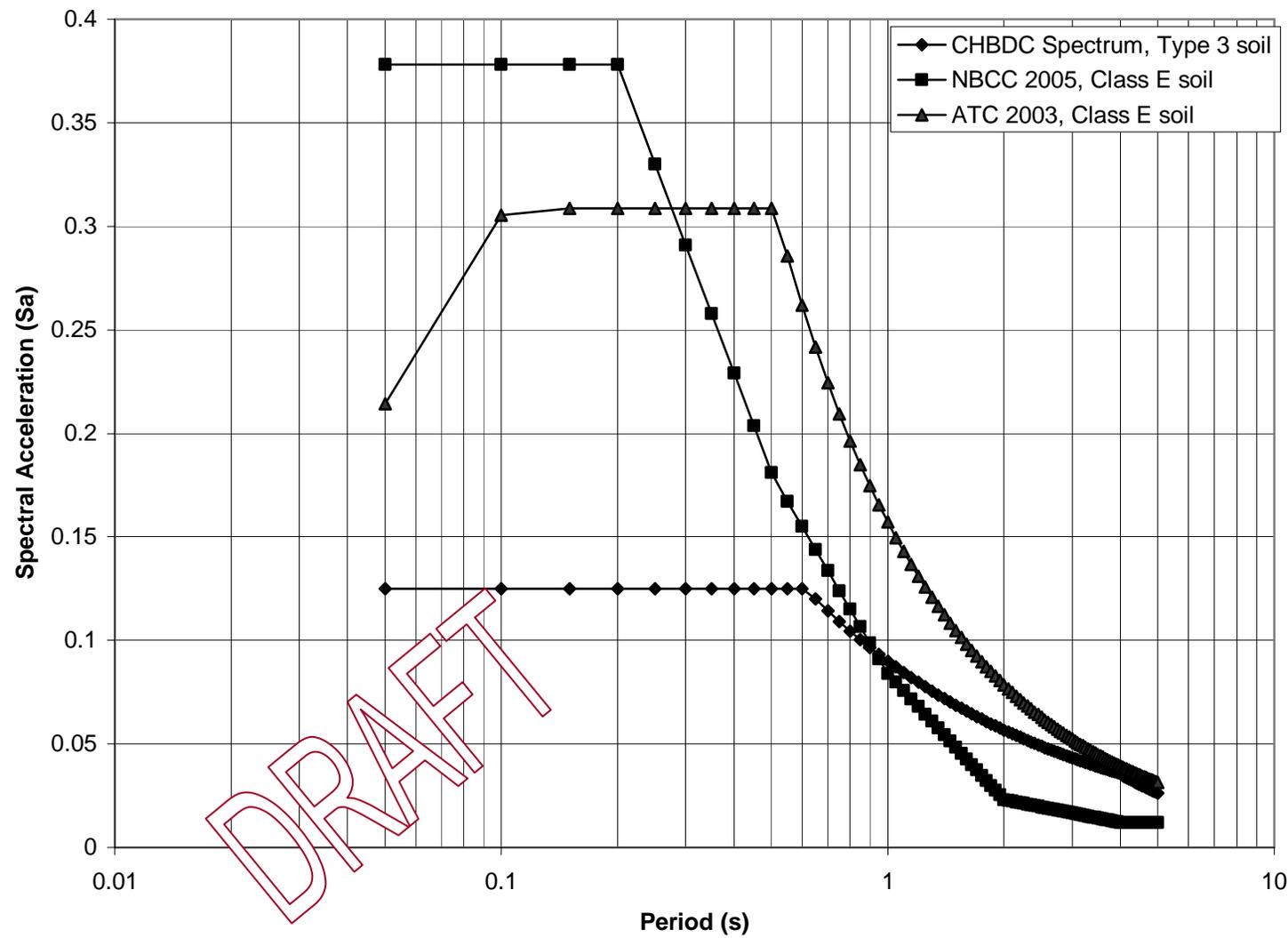
• Water Content Correlation  
 ■ Oedometer  
 — Design Line

- Notes:
- This figure is to be read with the accompanying reports "Foundation Design, Detroit River International Crossing, Bridge Approach Corridor, Part A", and "Foundation Design, Detroit River International Crossing, Bridge Approach Corridor, Part B" prepared by Golder Associates, February 2006.
  - Data and correlations used for development of this figure are discussed in the above referenced reports.
  - Limitations of the Design Lines are discussed in "Foundation Design, Detroit River International Crossing, Bridge Approach Corridor, Part B", prepared by Golder Associates, February 2006.
  - Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.

	SCALE	NOT TO SCALE
	DATE	FEB. 2007
FILE:	FILE NAME	CHECK
PROJECT NO:	04-1111-060	REVIEW
		DESIGN
		DRAW
		JWB
		JW
		FJH

<b>SUMMARY OF GEOTECHNICAL PROPERTIES BOREHOLE BH/FV/CPT-23</b>	
<b>DETROIT RIVER INTERNATIONAL CROSSING</b>	<b>FIGURE 1</b>

**Spectral Acceleration Curves (I = 1.0)  
Windsor, Ontario**



SCALE	As Shown
DATE	Feb 2007
DESIGN	JS
CAD	
CHECK	MS
REVIEW	

**SPECTRAL ACCELERATION CURVES**

FILE:	Spectral.ppt
PROJECT NO:	04-1111-060

DETROIT RIVER  
INTERNATIONAL CROSSING

FIGURE  
**2**

## **Appendix F: Evaluation Matrix**

Detroit River International Crossing

Bridge Evaluation Criteria Matrix

Screening Criteria:		Initial Cost		Constructability										Safety and Security										Retain		
Type Study Option	Construction Cost (\$000,000)	Cost Risk (Scale 1-5) <sup>1</sup>	Duration (months)	Schedule Risk (Scale 1-5)	Disruption (Scale 1-5)		Major Utilities			Contam. Sites <sup>4</sup>		Foundation Compatibility (Scale 1-5)	Technical Challenges (Scale 1-5)	Risk to Bridge (# Industries) <sup>3</sup>			Risk to Residents (No. DU's.)		Emergency Resp. (km)		Navigation Interference (m of river)	Vulnerability (Scale 1 - 5)				
					U.S.	Canada	# U.S.	# Canada	Risk (Scale 1-5)	#	Risk (Scale 1-5)			# U.S.	# Canada	Risk (Scale 1-5)	U.S.	Canada	U.S. <sup>2</sup>	Canada		Man-Made	Natural		Ship Impact	
<b>Crossing X10(A)</b>																										
Option 1	770	920	2	62	2	3	4	3	3	4	2	4	4	3	2	1	3	118	14	3.8	7.3	641	3	4	5	X
Option 2	680	810	4	56	2	3	4	3	3	4	2	4	4	3	2	1	3	118	14	3.8	7.3	641	3	4	3	
Option 3	620	740	1	55	2	3	4	3	3	4	2	4	4	2	2	1	3	118	14	3.8	7.3	641	3	4	3	
<b>Crossing X10(B)</b>																										
Option 4	430	510	2	51	4	3	3	3	1	4	2	4	4	2	1	1	3	131	10	3.6	7.9	338	3	4	5	X
Option 5	370	440	3	43	3	3	3	3	1	4	2	4	4	3	1	1	3	131	10	3.6	7.9	338	3	4	3	
Option 6	480	550	5	52	4	3	3	3	1	4	2	4	4	3	1	1	3	131	10	3.6	7.9	338	3	4	5	X
Option 7	470	540	5	49	4	3	3	3	1	4	2	4	4	3	1	1	3	131	10	3.6	7.9	338	3	4	5	
Option 8	420	490	4	43	3	3	3	3	1	4	2	4	4	3	1	1	3	131	10	3.6	7.9	338	3	4	3	
<b>Crossing X11(C)</b>																										
Option 9	450	530	3	47	5	4	2	1	1	4	5	4	4	3	1	1	2	102	600	1.0	9.1	291	2	4	5	X
Option 10	500	580	5	42	4	4	2	1	1	4	5	4	4	3	1	1	2	102	600	1.0	9.1	291	2	4	5	X
Option 11	520	600	5	51	4	4	2	1	1	4	5	4	4	3	1	1	2	102	600	1.0	9.1	291	2	4	5	

Notes:

- Evaluation process and factors are as outlined in the July 2006 Technical Memo.
- All scale factors (1 - 5) are from most to least (i.e., most risk = 1 and least risk = 5), or from worst to best.
  - Emergency Response; for X10(A&B) uses interchange Option 1 and Plaza 4; for X11(C) uses interchange Option 1 and Plaza 5.
  - Industries are only considered if they are major industries presenting a potential risk to the structure.
  - U.S. EPA, MDEQ registered sites, plus Revere Copper and Solvay.

## **Appendix G: Documents Incorporated by Reference**

- [1] NTH, *Preliminary Draft Geotechnical Evaluation Report, Crossings X-10 and X-11, United States Shoreline*, September 21, 2006.
- [2] Parsons, Technical Memo, *River Crossing Bridge Cross Section*, June 29, 2006.
- [3] U.S. Army Corps of Engineers, Institute for Water Resources, Navigation Data Center, *Port Series No. 45*, 1996, pp. 7-12, 33-34.
- [4] Corradino Group, Memorandum, *Preliminary Airspace Evaluation: Detroit International River Crossing (DRIC) Bridge Structure*, Oct. 18, 2006.
- [5] Parsons, Technical Memo, *Construction Schedule*, December 29, 2006.
- [6] Parsons, Technical Memo, *Evaluation of Practical Bridge Options*, July 5, 2006.