LASALLE CAUSEWAY BASCULE BRIDGE

COUNTERWEIGHT REHABILITATION STUDY

Technical Memorandum | DRAFT

PSPC Project No. R.108246.001

March 31, 2021

PREPARED FOR:

Public Works and Government
Services Canada

wsp

REVISION HISTORY

ISSUANCE APPROVAL

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REVISION HISTORY

NOTE: Revisions are identified with a revision bar unless otherwise specified below. Any printed copies are uncontrolled and may not be current. Revision of any appendices are as per listed on the appendix itself.

Rev	Date	Description	Prepared By	Reviewed By	Approved By	Client Approval
1	Mar. 31, 2021	Draft Report	JB/RA	MM	MM	

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TABLES

FIGURES

APPENDICES

No table of contents entries found.

1 INTRODUCTION

1.1 BACKGROUND

The Lasalle Causeway is located in Kingston, Ontario and forms part of Highway #2, crossing the Cataraqui River at the entrance to the Kingston Harbour from Lake Ontario. The causeway provides a significant contribution to the socio-economic operations of the City of Kingston as 25,000 to 28,000 vehicles cross it daily. Open to the public in 1917, the Bascule Bridge Counterweight is the subject of this investigation.

The existing concrete counterweight weighs approximately 550 tonnes and was part of the original construction in 1917 and is suspended from the counterweight truss above the roadway. The steel truss sections extend into the center of concrete and acts as the main supports for the mass of concrete, while steel bars and wire mesh provide support to the external faces of the concrete. Except at the North and South faces of the Counterweight, all other faces are covered with what appear to be corrugated metal roofing panels. There are steel plates mounted on the North and South faces of the Counterweight which are secured in place by threaded steel rods. No known repairs have been conducted to the Counterweight in the past.

Concrete cores were extracted from the Bascule Bridge counterweight as part of the 2018 Comprehensive Detailed Inspection (CDI) for observation and testing. A total of six 100 mm diameter cores ranging in depth from 126 mm to 611 mm were taken from the East and West faces of the Counterweight. It was only possible to test one of the six cores for compressive strength and the resulting test determined a compressive strength of 11.9 MPa at an approximate depth of 50 mm to 255 mm. Based on the Ministry of Transportation Ontario (MTO) Structure Rehabilitation Manual, structural concrete with compressive strengths under 20 MPa is of poor quality. Further investigation limited to a visual inspection of the counterweight's visible elements was completed by Parsons as part of the 2019 CDI. The 2019 CDI indicated the concrete was generally in poor condition exhibiting disintegration, spalling, efflorescence with and without stalactites and stalagmites. There were areas of exposed and corroding reinforcing steel and wire mesh. The interior space of the Counterweight which was an empty chamber was sounded with a hammer, where many areas were hollow or soft sounding indicating deep concrete disintegration. Through inspection of the Counterweight lower angle area, specifically the gap between the sheet metal cladding and counterweight, varying amounts of debris bearing on the angles and soffit cladding were observed and later removed.

1.2 PURPOSE

The purpose of this technical memorandum is to summarize the structural assessment and evaluation of the Concrete Counterweight system and provide recommendations for rehabilitation strategies that should be further developed in Stage 2 of the Counterweight study.

The goal is to determine and then develop the subsequent rehabilitation strategies that will achieve a Structural Condition rating of fair (4) in accordance with the Bridge Inspection Manual (BIM) for all components of the Counterweight.

1.3 SCOPE

The scope of the technical memo is to determine possible short-term a long-term rehabilitation options for the Counterweight system (embedded Steel Truss and Concrete Counterweight). This includes the following:

- Structural assessment and evaluation of Concrete Counterweight and Steel Counterweight truss;
- Establish weight of Concrete Counterweight System such that balancing is maintained;

2 COUNTERWEIGHT ASSESSMENT

To determine possible short-term and long-term rehabilitation options of the Concrete Counterweight System that would achieve a 25-year service life and a Structural Condition Rating of fair (4) in accordance with the Bridge Inspection Manual (BIM), the results of the field investigation were analyzed to assess the overall condition of the Counterweight System.

Overall, the condition of the concrete at the surface of the Counterweight was found to be in poor condition, showing significant delamination, spalling, cracks, and section loss. This severe damage extended to a depth of 100 mm – 300 mm in some locations, with the compressive strength of the concrete found at the surface was found to be a maximum of 10.4 MPa. Further coring results excessive cracking within the first 1300 mm, after which sound concrete is reached, while carbonation depths of 90 mm – 115 mm were measured on site using a Phenolphthalein solution. Therefore, to ensure future stability of the Counterweight and public safety, WSP is recommending any potential short-term or long-term rehabilitation option includes the removal of at least 600 mm of concrete from all faces of the Counterweight. The initial 300 mm of removals will address the severe damage that extends into the concrete, while the subsequent 300 mm of removals ensures any severe damage that remains is removed, all loose concrete and/or aggregates are removed and a proper bonding surface is provided for when new concrete is placed.

Furthermore, in addition to the Counterweight concrete inspection, the embedded structural steel ('Girder F' on the original contract drawings) that is exposed within the four (4) access hatches on the top face were also inspected. It was noted that these steel truss members displayed severe deterioration (approximately 30% - 50% section loss) due to their exposure to atmospheric conditions and moisture. To determine any potential rehabilitation recommendations for these members, a section loss of 50% will be assigned to these members for the structural analysis. Other embedded steel members exposed during the field investigation were noted to have surface rust but no significant loss of section, which is similar to previous inspections. Therefore, a section loss of 5% will be assigned to the remaining embedded steel members for the structural analysis.

Given the above assessment and recommendations, the following sections are potential immediate, short- and long-term rehabilitation options for the Counterweight.

2.1 IMMEDIATE REPAIR STRATEGY

During the field investigation of the concrete Counterweight, it was seen there was significant advanced deterioration of the concrete surface (delamination, disintegration and spalls), with portions of the concrete being susceptible to detach from the Counterweight. While the existing Counterweight cladding has previously prevented small amounts of debris from falling on to the roadway surface, the cladding is not structural in nature and therefore if large pieces of concrete were to detach from the Counterweight, it may break through the cladding and fall on to the roadway surface below. The advanced deterioration of the concrete surface posses a potential safety risk to the public (specifically the more advanced deterioration on the underside), created a need to an interim repair strategy, prior to the implementation of either a short-term or long-term rehabilitation program. Therefore, WSP recommended and subsequently designed and Interim Repair strategy for the concrete Counterweight that is to be implemented prior to the 2021 navigation opening.

The Interim Repair strategy was designed for a service life of 0-5 years, therefore, the short-term or long-term rehabilitation strategy shall be implemented by PSPC within that timeframe.

2.2 OPTION 1 – REPLACE ENTIRE COUNTERWEIGHT

Given the PSPC requirement for a long-term rehabilitation strategy, for the first rehabilitation option, WSP is recommending the removal and replacement of the entire Concrete Counterweight. The extent of this rehabilitation strategy can provide PSPC the flexibility and freedom to replace the Counterweight in kind or provide a new configuration. Furthermore, even though Cores C6 and C7 that were removed from the top face of the Counterweight revealed the structural steel embedded within the counterweight exhibited very little deterioration, the exposure of the steel truss will present an opportunity for a full detailed inspection of all steel members. In this case, WSP also recommends that embedded steel truss ('Girder F'), which was noted to have severe section during the field investigation, be rehabilitated given the possibility for further damage to these members during concrete removal operations.

PSPC would be able to address any additional steel members or components that may require intervention, therefore, proving a minimum service life of 25 years for the entire Counterweight System (concrete and steel truss).

Considering the PSPC serviceability requirements (25 years) for the rehabilitation, Option 1 may be overly aggressive, as the rehabilitated Counterweight system may outlive the remainder of the 104-year-old structure. However, given the overall flexibility and age of the Counterweight components, WSP proceeded with an overall assessment, evaluation, and balancing of the structure to determine the feasibility of this rehabilitation option. Given the current stage of the project, the structural assessment, evaluation, and balancing was done under the assumption that the Counterweight would be replaced in kind.

2.3 OPTION 2A – PARTIAL REMOVAL/REPLACEMENT OF CONCRETE

The second rehabilitation option WSP is recommending is the removal of 600 mm of concrete from all faces of the of the concrete Counterweight and replacement in kind. Given the field investigation results noted above, the removal of 600 mm of concrete should eliminate all deteriorate, loose, and/or aggregates from the Counterweight and would expose sound concrete for adequate bonding.

It was determined this strategy may be the most efficient option, as it reduces the amount of required removals, while protecting the remaining concrete and embedded steel truss by encapsulating it in a new reinforced concrete shell. While the details of the new reinforced concrete details would be determined during detailed design, for the purposes of this memo, it is being assumed the new reinforced concrete shell will be self sustaining.

Further details for this rehabilitation option will be developed in Stage 2 of the project, which will include the use of steel dowels to connect the new and existing concrete, the use of a galvanic anode system due to the presence of chlorides, and the use of premium reinforcing bars, such as stainless steel.

2.4 OPTION 2B – STAGED PARTIAL REMOVAL/REPLACEMENT OF CONCRETE

Option 2b utilizes the same repair strategy of Option 2a; removal of 600 mm of concrete and replacement in kind, however, Option 2b would be done in stages. This will provide additional flexibility as it may be required to stage the rehabilitation strategy to mitigate traffic impacts on the City of Kingston.

2.5 OPTION 3 – SHOTCRETE APPLICATION

The application of shotcrete to the existing concrete Counterweight was initially considered as a potential rehabilitation option prior to the field investigation, due to its commonness in the construction industry. Following the field investigation, it was quickly determined for numerous reasons, that a shotcrete shell is not feasible.

Even if deteriorated concrete was removed from the existing concrete Counterweight to provide a sound bonding surface, the amount of shotcrete applied would need to be precise to ensure balancing of the structure. In this regard, the use of formwork system would provide a more accurate final weight of concrete added to the Counterweight. therefore, shotcrete application was not considered in the structural assessment and evaluation, and the balancing.

3 STRUCTURAL EVALUATION

A structural evaluation has been conducted to assess the stability of the through truss, Counterweight truss and embedded truss during construction of the Counterweight rehabilitation options. For this evaluation, three-dimensional models were developed and analyzed using the commercially available finite element modeling software CSiBridge (v22). Of the rehabilitation options previously discussed, Options 1, 2A and 2B were assessed to determine the affect of the concrete removals on the structure. A base model representing the existing conditions of the structure was also evaluated. For each model, the structure was modeled in its closed position with vertical restraints at the toe to prevent uplift. All components of the structure were modeled with the section and material properties provided in the original contact drawings and past rehabilitation drawings where applicable, except for members in the embedded truss, as noted in **Section 2.**

For each rehabilitation option, the main span truss members were evaluated under ultimate limit states using load factors provided in Section 13 of the Canadian Highway Bridge Design Code (CSA S6-19) for rehabilitation design. The results provided herein have been used to assess the feasibility of each rehabilitation option, including any potential requirements for strengthening or traffic restrictions during construction, such as load postings.

In addition to the structural evaluation, a span balance analysis was performed. A properly balanced bascule span is important to ensure the bridge operates properly and does not overload the machinery or structure. The existing counterweight has balance blocks located in the counterweight pockets and balance plates mounted to the counterweight truss back arm, which are available for adjusting the imbalance. The analysis was performed for each option to establish the anticipated final weight of Concrete Counterweight System such that balancing is maintained. Only the final balance was considered, as it is anticipated the construction work will be completed during navigational channel outages with the bridge locked in the closed position.

3.1 OPTION 1 – REPLACE ENTIRE COUNTERWEIGHT

Option 1, complete removable of concrete counterweight is considered in the process of rehabilitation of counterweight. For this considered case, the trough truss, counterweight truss and embedded truss are modelled in CSiBridge software with the entirety of the concrete removed from the Counterweight and the isometric view of the same model is presented in Figure 1.

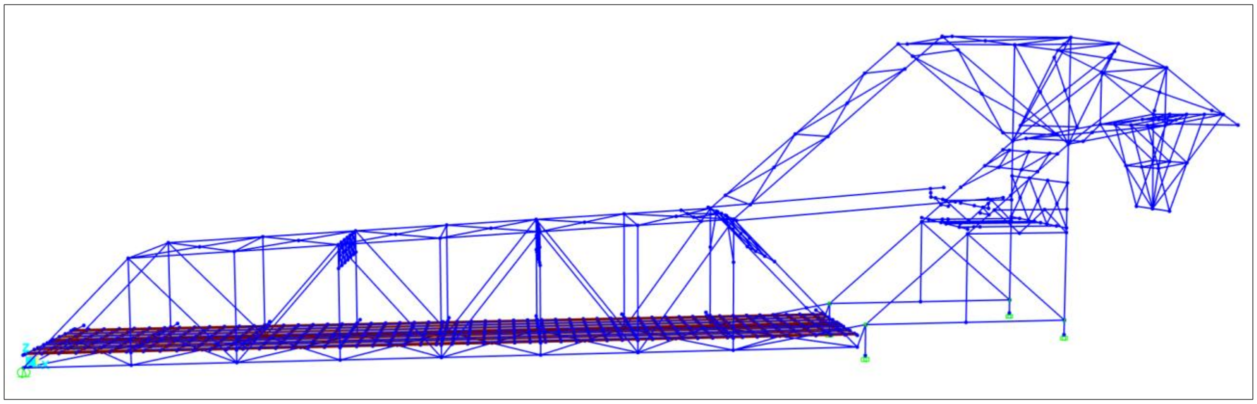

Figure 1: CSiBridge Model of the Bridge – Option 1

The load and load combination are considered as per the Section 13 of CSA S6-19 and the design checks are performed to evaluate the Demand/Capacity (D/C) ratio of the individual structural element of the bridge. The section loss in each structural elements/structural group are considered as per what is noted in **Section 2**. For design checks, the bridge is further grouped into three categories: such as Trough truss, Counterweight truss and Embedded truss, the summary results are presented separately for each group for clarity.

3.1.1 THROUGH TRUSS

The D/C ratio was evaluated for the through truss members and the results show there are multiple members where the demand on the member exceeds their capacity (D/C >1). The members with inadequacy along with the members ID's of the through truss members are presented in Figure 2.

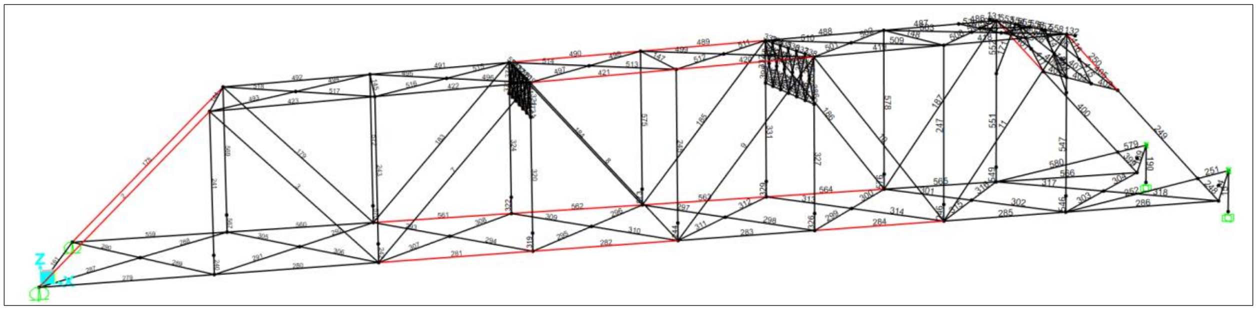

Figure 2: Option 1 – Through Truss – Elements with D/C >1 shown in red.

The governing D/C Demand vs capacity comparison for compression and tension are presented in **Table 3-1**.

Table 3-1: Governing D/C for Through Truss Members – Option 1

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
489	Section 01	04 TT Top chord	2140.9	2460.8	1.15	Compression Demand/Capacity
561	Section 02	03 TT Bottom chord	2039	2272	1.11	Tension Demand/Capacity

3.1.2 COUNTERWEIGHT TRUSS

The D/C demand and capacity ratio were evaluated for the Counterweight truss members and the results determined that all members have a capacity that exceeds the demand (D/C<1)found that all members capacity for the demand of this optional study are adequate and are therefore support all loads likely to be applied to them if all concrete were to be removed from the Counterweight. The Counterweight truss along with the member ID's are presented in the **Figure 3**.

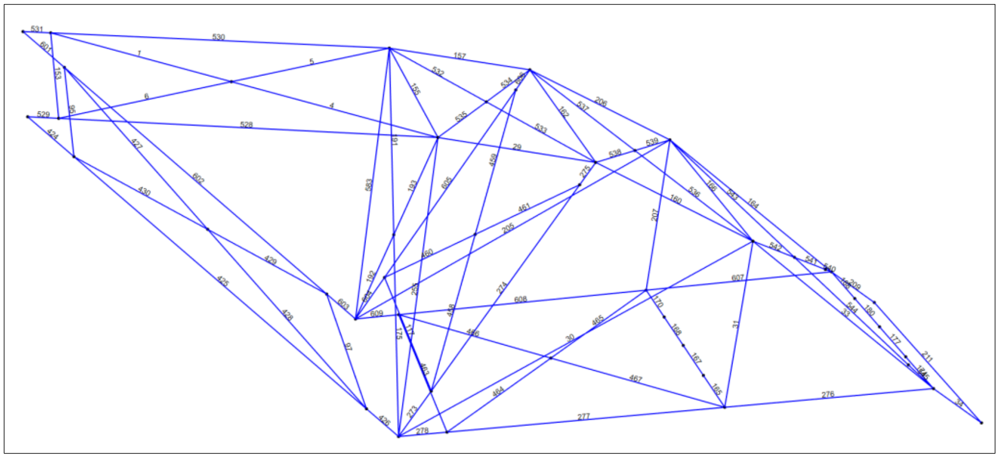

Figure 3: Option 1 – Counterweight Truss – Member ID's.

The governing D/C vs capacity comparison for compression and tension are presented in **Table 3-2**.

Table 3-2: Governing D/C for Counterweight truss members – Option 1

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
601	Section 16	10 CTWT Diagonal	4700	346	0.07	Compression Demand/Capacity
531	Section 22	08 CTWT Top Chord	7385	463	0.06	Tension Demand/Capacity

3.1.3 EMBEDDED TRUSS

This option study's the behavior of bridge with no Counterweight concrete, which the embedded truss will not experience any considerable load, therefore all capacities are adequate and the D/C ratios are approximately zero (0). The embedded truss along with the member ID's are presented in the **Figure 4**.

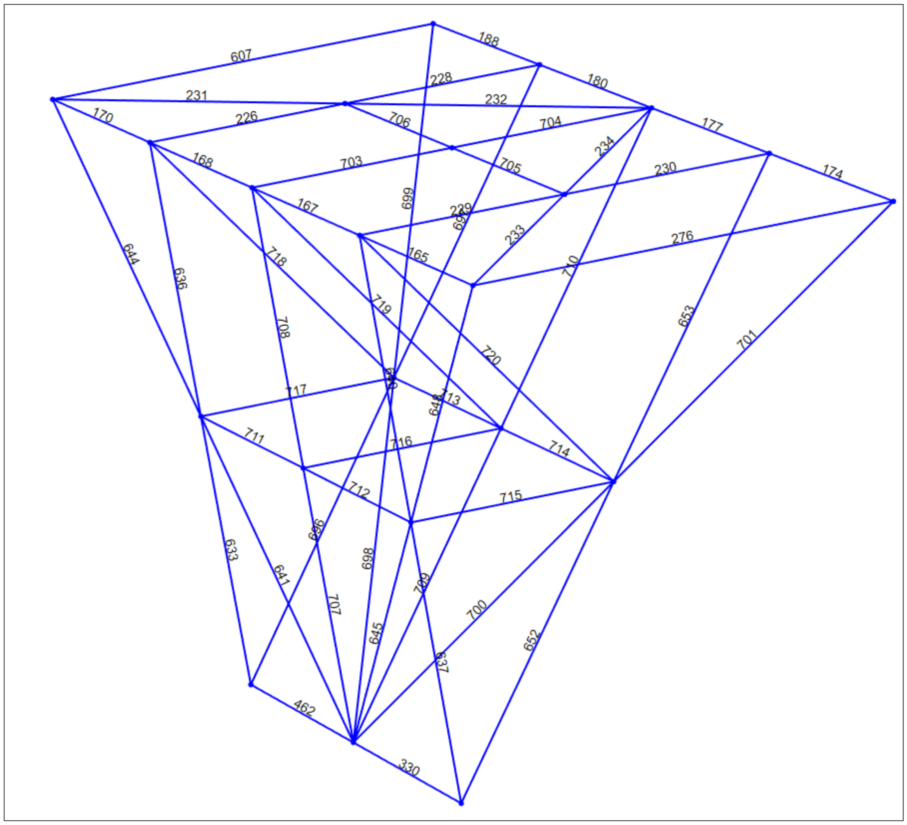

Figure 4: Option 1 – Embedded Truss – Member ID's.

3.1.4 RESULTS AND DISCUSSION

The D/C of the Through truss, Counterweight truss, and Embedded truss structural members were evaluated for the Option 1 and it was found that several members of the Through truss had a D/C > 1, while all members of the Counterweight truss and Embedded truss had a D/C < 1. Therefore, to further develop Option 1 in the conceptual design phase, additional evaluation is recommended as per Section 14 of CSA S6-19 with consideration given to load rating.

3.2 OPTION 2A - PARTIAL REMOVAL/REPLACEMENT OF CONCRETE

Option 2, the partial removal of 600 mm of concrete from all faces of the concrete Counterweight was analyzed using CSiBridge in the closed position. The model included the Through truss, Counterweight truss, Embedded truss, and remaining concrete after removals. The isometric view of the model is presented in **Figure 5**.

The load and load combination are considered as per the section 13 of CSA S6-19 and the design checks are performed to evaluate the D/C ratio of the individual structural element of the bridge. The section loss in each structural elements/structural group are considered as per what is noted in **Section 2**. The Bridge is further grouped into three categories: Through truss, Counterweight truss and Embedded truss, the summary results are presented separately for each group.

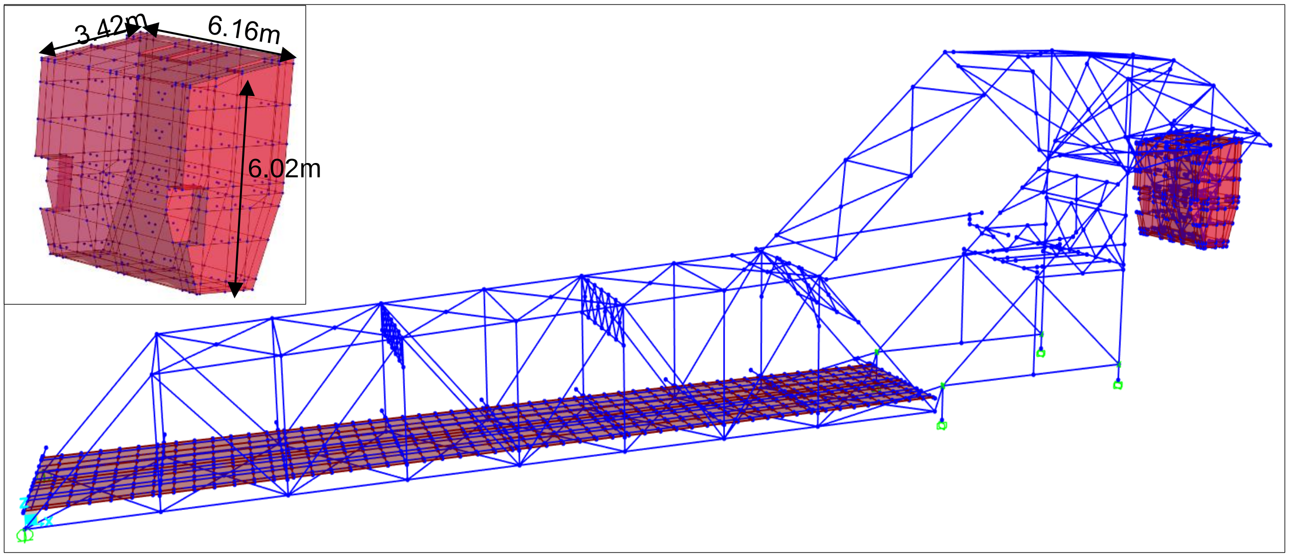

Figure 5: CSiBridge Model of the Bridge – Option 2A Counterweight

3.2.1 THROUGH TRUSS

The D/C ratio was evaluated for the Through truss members and the results show that all members have a D/C < 1. The governing D/C ratios for compression and tension are presented in **Table 3-3**.

Table 3-3: Governing D/C for Through truss members – Option 2A

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
186	Section 07	01 TT diagonal	1345.9	1261.9	0.94	Compression Demand/Capacity
187	Section 07	01 TT diagonal	2035	1557	0.77	Tension Demand/Capacity

3.2.2 COUNTERWEIGHT TRUSS

The D/C ratio was evaluated for the Counterweight truss members and the results show that all members have a D/C < 1. The governing D/C ratios for compression and tension are presented in **Table 3-4**.

Table 3-4: Governing D/C for Counterweight Truss Members – Option 2A

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
604	Section 17	10 CTWT Diagonal	1950	605	0.31	Compression Demand/Capacity
206	Section 24	08 CTWT Top Chord	6082	1942	0.32	Tension Demand/Capacity

3.2.3 EMBEDDED TRUSS

As with any model that incorporates both linear and solid finite elements the common points where the constraints of the two types of elements are joined provide the continuity between the steel frame (linear elements) and the concrete (solid elements). The nature of the stiffness matrix controls that loads are transferred between the materials at the common points and thus the mesh density and element segments have to be chosen to ensure a representative transfer. The frame shown in **Figure 6** below, used to establish the geometry was split and constrained to the concrete and a sensitivity analysis was completed by assigning the translation constraints and translation/rotational joint constraints between the Embedded truss and the concrete Counterweight. This ensures that while assumptions influence the results, the assumptions do not dominate or change the accuracy of the results.

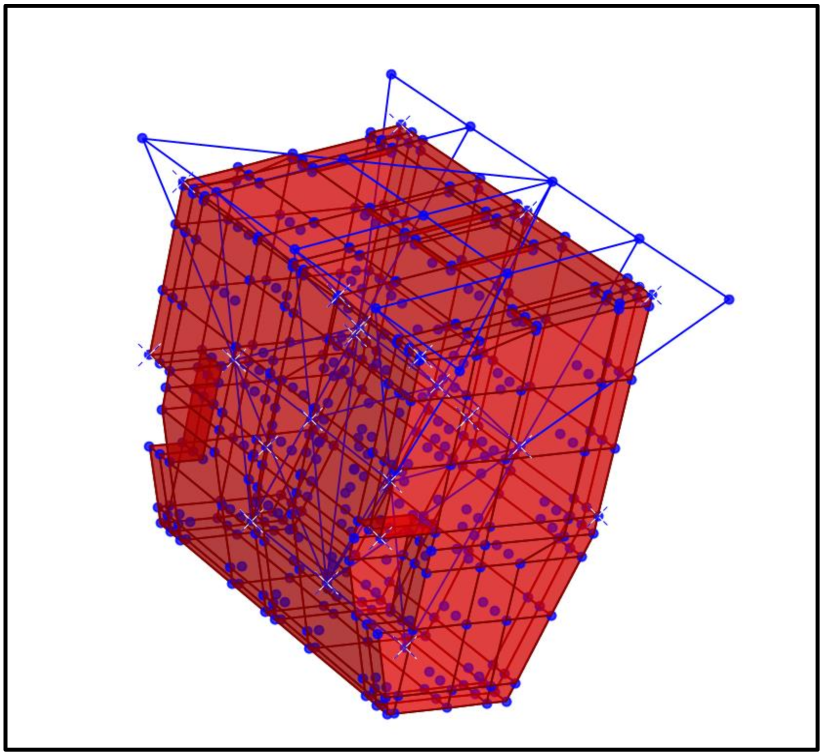

Figure 6: Location of Joint Constraints in the Embedded Steel and Concrete Counterweight

The D/C ratio was evaluated for the Embedded truss members and the results show that all members have a D/C < 1. The governing D/C ratios for compression, tension, and moment with translation joint constraint are presented in **Table 3-5** and with both translation and rotational constraints are presented in **Table 3-6**.

Table 3-5: Governing D/C for Embedded Truss Members – Option 2A (with Translation constraints)

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
170	Section 58	CTWT Embed Truss	969	412.28	0.43	Compression Demand/Capacity
648	Section 75	CTWT Embed Truss	2883.16	1406.46	0.49	Tension Demand/Capacity
648	Section 75	CTWT Embed Truss	61.65	3.46	0.49	Moment Mx Demand/Capacity
233	Section 74	CTWT Embed Truss	171.43	36.64	0.21	Moment My Demand/Capacity

Table 3-6: Governing D/C for Embedded Truss Members – Option 2A (with Translation and rotational constraints)

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
170	Section 58	CTWT Embed Truss	969	421.17	0.43	Compression Demand/Capacity
700	Section 68	CTWT Embed Truss	617.81	399.65	0.65	Tension Demand/Capacity
713	Section 68	CTWT Embed Truss	20.61	9.33	0.45	Moment Mx Demand/Capacity
233	Section 74	CTWT Embed Truss	171.43	41.71	0.24	Moment My Demand/Capacity

3.2.4 RESULTS AND DISCUSSION

The D/C of the Through truss, Counterweight truss, and Embedded truss structural members were evaluated for the Option 2a and it was found that all members had a D/C < 1.

3.3 OPTION 2B - STAGED PARTIAL REMOVAL/REPLACEMENT OF CONCRETE

Option 2b, the partial removal of 600 mm of concrete from all faces of the concrete Counterweight in stages, was analyzed using CSiBridge in the closed position. The model included the Through truss, Counterweight truss, Embedded truss, and remaining concrete after removals. The isometric view of the model is presented in **Figure 7**.

The load and load combination are considered as per the section 13 of CSA S6-19 and the design checks are performed to evaluate the D/C ratio of the individual structural element of the bridge. The section loss in each structural elements/structural group are considered as per what is noted in **Section 2.** The Bridge is further grouped into three categories: Through truss, Counterweight truss and Embedded truss, the summary results are presented separately for each group.

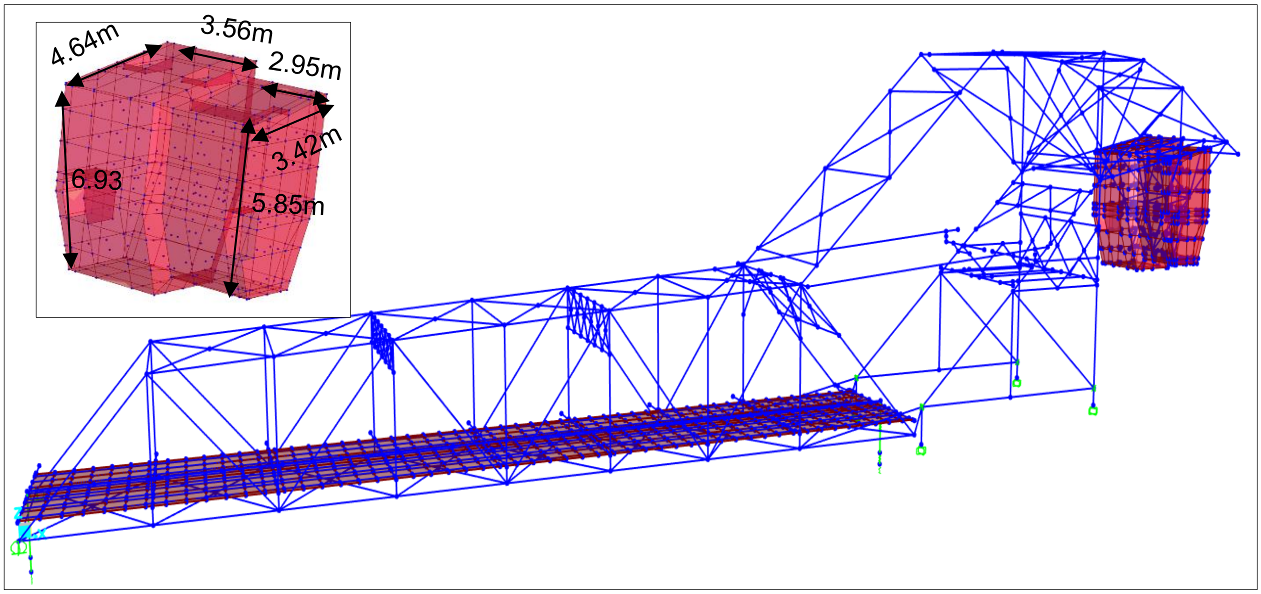

Figure 7: CSi Model of the Bridge – Option 2B Counterweight

3.3.1 THROUGH TRUSS

The D/C ratio was evaluated for the Through truss members and the results show that all members have a D/C < 1. The governing D/C ratios for compression and tension are presented in **Table 3-7.**

Table 3-7: Governing D/C for Through Truss members – Option 2B

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
186	Section 07	01 TT diagonal	1345.9	1313	0.94	Compression Demand/Capacity
187	Section 07	01 TT diagonal	2035	1638	0.80	Tension Demand/Capacity

3.3.2 COUNTERWEIGHT TRUSS

The D/C ratio was evaluated for the Counterweight truss members and the results show that all members have a D/C < 1. The governing D/C ratios for compression and tension are presented in **Table 3-8**.

Table 3-8: Governing D/C for Counterweight Truss members – Option 2B

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
604	Section 17	10 CTWT Diagonal	1950	810	0.42	Compression Demand/Capacity
206	Section 24	08 CTWT Top Chord	6082	2513	0.41	Tension Demand/Capacity

3.3.3 EMBEDDED TRUSS

Similar to Option 2a, a sensitivity analysis was completed by assigning the translation constraints and translation/rotational joint constraints between the Embedded truss and the concrete Counterweight. For the analysis of the Embedded truss with translation constraints, all members have a D/C < 1. However, various members have a D/C > 1 with translation and rotational joint constraints and are shown in **Figure 8**.

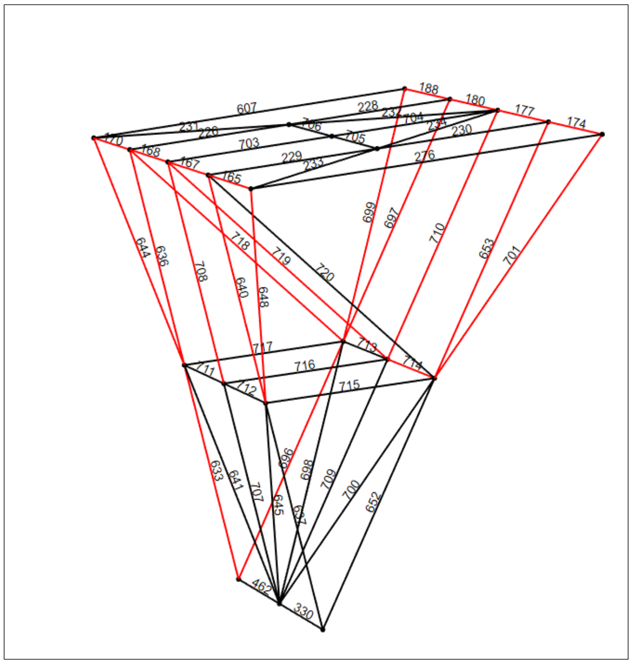

Figure 8: Option 2B – Embedded Truss – Members with D/C > 1 shown in red (with translation/rotational constraint)

The governing D/C ratios for compression, tension and moment with translation joint constraint are presented in **Table 3-9** and with both translation and rotational constraints are presented in **Table 3-10.**

Table 3-9: Governing D/C for Embedded truss members – Option 2B (with translation constraint)

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
170	Section 58	CTWT Embed Truss	969	579.69	0.60	Compression Demand/Capacity
703	Section 64	CTWT Embed Truss	379.37	293.08	0.77	Tension Demand/Capacity
188	Section 59	CTWT Embed Truss	21.55	6.24	0.29	Moment Mx Demand/Capacity
233	section 74	CTWT Embed Truss	171.43	32.05	0.19	Moment My Demand/Capacity

Table 3-10: Governing D/C for Embedded truss members – Option 2B (with translation and rotational constraint)

MEMBER ID	SECTION	ELEMENT GROUP	MINIMUM CAPACITY (KN)	DEMAND (KN)	D/C	DESIGN MODE/ACTION
170	Section 58	CTWT Embed Truss	969	674.78	0.70	Compression Demand/Capacity
696	Section 69	CTWT Embed Truss	540.58	678.42	1.25	Tension Demand/Capacity
188	Section 59	CTWT Embed Truss	21.55	63.28	2.94	Moment Mx Demand/Capacity
708	Section 66	CTWT Embed Truss	23.82	59.32	2.49	Moment My Demand/Capacity

3.3.4 RESULTS AND DISCUSSION

The D/C of the Through truss, Counterweight truss, and Embedded truss structural members were evaluated for the Option 2b and it was found that all members had a D/C < 1 for the case with translation constraint, however, various members had a D/C > 1 with for the case with translation and rotational constraint, which is most likely due to the load imbalance. Therefore, to further develop Option 2b in the conceptual design phase, additional investigation and potential load rating, as per Section 14 of CSA S6-19, may be required.

4 BALANCING

An analysis was performed to determine if the existing bridge balance can be maintained without requiring design modifications to the existing counterweight after concrete replacement is completed. This included analyzing the imbalance moment over the entire range of the bridge operation. This is especially important for a Strauss-Type Trunnion Bascule Bridge, as the effect of the operating strut weight on the bridge balance varies throughout the bridge operation.

4.1 BALANCE ANALYSIS METHOD

Counterweight replacement options 1, 2A, and 2B were each analyzed for the anticipated final span balance condition. The final span balance condition is based on the existing balance condition and available balance blocks and plates. If the bridge balance cannot be maintained through adding or removing the available balance blocks or plates, the counterweight design will require modifications (such as increasing the counterweight pocket size). Maintaining the balance bridge condition will limit the loading on the span drive machinery during operation.

The span balance analysis is based on the existing 2018 span balance report by Stafford Bandlow Engineering. The estimated change in weights for each concrete replacement option was added to the existing balance condition to determine the new balance condition. From there, an example final balance condition was calculated based on the available adjustable ballast. Per the study field investigations, it is assumed that all counterweight pockets are completely full in addition to the available rear steel balance plate.

It is anticipated all work for each option will be performed during navigational channel outages with the bridge in the closed position. If it is later determined the bridge will be required to operate during construction work, an interim balance will be required and calculated during final design.

CSA specifications do not provide specific requirements for the span imbalance moment; so for this study the American AASHTO Movable Bridge Specification Second Edition Requirements are used. AASHTO recommends single leaf bascule bridges to be toe heavy up to a maximum toe reaction of 1,000 lb. (4,448 N) per bascule girder with the leaf fully seated. If the bridge can be balanced to the AASHTO allowable toe reaction through balance blocks/plates adjustments alone, no modifications to the counterweight are required.

Additional requirements are also given in AASHTO for the alpha angle. The alpha angle locates the span center of gravity throughout the bridge operation. AASHTO's requirements are for typical trunnion bascule bridges and may not be ideal for a Strauss-Type bascule bridges due to the effect of the operating strut.

During bridge operation, the imbalance moment from the operating strut varies differently from the rest of the bridge balance. As such, the operating strut effect is accounted for separately in the calculations. If the bridge can be balanced to maintain the existing alpha angle in addition to the toe reaction requirements, no modifications to the counterweight design are required.

4.2 OPTION 1

The scope for option 1 includes the complete replacement of all concrete in the counterweight. The replaced concrete has a higher density (2,400 kg/m^3) than the existing concrete as-measured density (2,200 kg/m^3). This will increase the total weight of the counterweight after concrete replacement. Additionally, the existing metal cladding will be removed and not reinstalled. In total, the change in weight including the concrete replacement and removal of the existing cladding is an additional 39,000 kg added to the counterweight.

The total weight of adjustable balance blocks on the existing span is approximately 31,000 kg. If all balance blocks and plates were removed when the concrete is replaced, there would still be a net total 8,000 kg added to the counterweight. span would become counterweight heavy and the alpha angle would not be maintained. This is shown in the table and graph below.

Modifications to the counterweight design would be required to accommodate the additional weight added to the counterweight. Modifications would include increasing the counterweight pocket area which will reduce the weight of the counterweight. Additional weight can also be added to the lift span arm; however, this is not recommended as it would increase the total loading on the trunnion pins.

Table 4-1: Option 1 - Balance Summary without Counterweight Modifications)

	MX	WR	STRUT EFFECT	TOE REACTION	ALPHA ANGLE
	(N*M)	(N*M)	(N*M)	(N)	(Degrees)
Initial Condition	-246,126	1,645,936	360,697	2,349	-98.6
Recommended Acceptable*	-	-		0 to 8896	-97 to -100
Final Balance	-567,188	2,213,438		**-4,234**	**-104.8**

*Recommended Acceptable Range is based on AASHTO requirements and the existing imbalance condition.

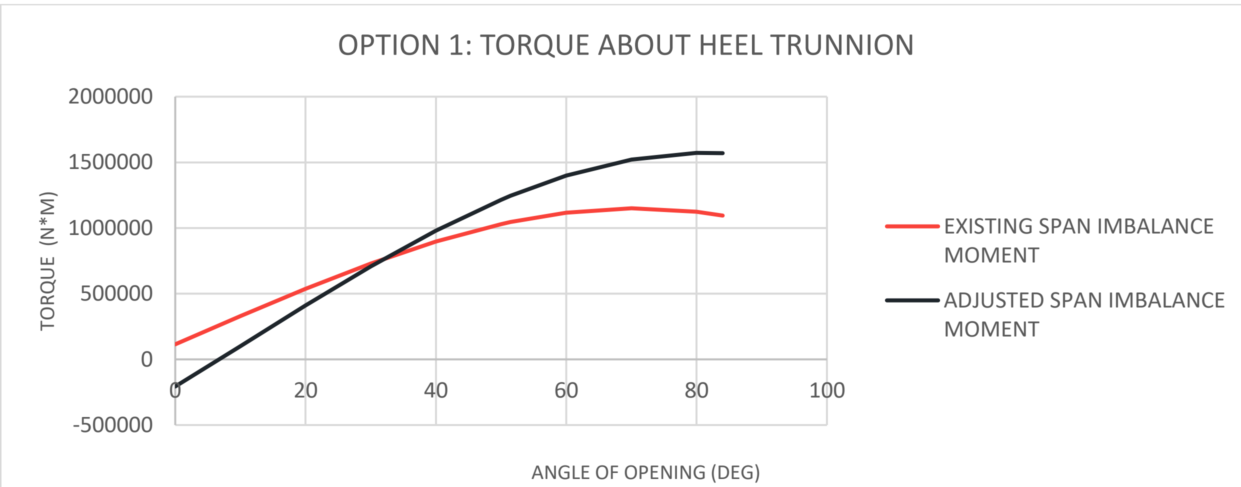

Figure 9: Option 1: Torque about Hell Trunnion

4.3 OPTION 2A

The scope for option 2 includes the replacement of the deteriorate concrete exterior layers only while leaving the remaining interior concrete in place. As noted in option 1, the new concrete density (2,400 kg/m^3) is heavier than the existing concrete as-measured density (2,200 kg/m^3). Additionally, the existing metal cladding will be removed and not reinstalled. The net change in concrete weight would be an additional 18,290 kg heavier.

There is 31,000 kg of existing adjustable balance blocks and plates. The existing adjustable ballast can completely account for the added weight from the concrete replacement. Therefore, no modifications to the existing counterweight design are required.

An example final balance has been calculated for the study. In the example, the toe reaction will decrease slightly but is within the AASHTO acceptable range. Additionally, the alpha angle will be maintained during the adjustments. The span balance results are shown in the table and graph below.

Table 4-2: Option 2A – Balance Summary

	MX	WR	STRUT EFFECT	TOE REACTION	A ANGLE
	(N*M)	(N*M)	(N*M)	(N)	(Degrees)
Initial Condition	-246,126	1,645,936	360,697	2,349	-98.6
Recommended Acceptable*	-	-		0 to 8896	-97 to -100
Final	-250,222	1,755,025		**2,265**	**-98.2**

*Recommended Acceptable Range is based on AASHTO requirements and the existing imbalance condition.

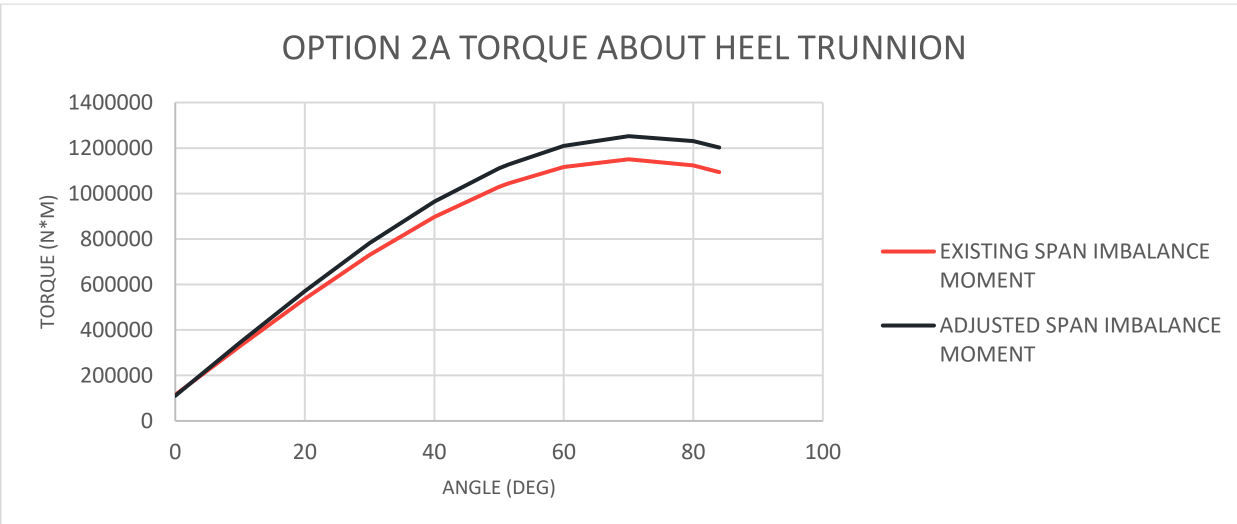

Figure 10: Option 2A – Torque about Heel Trunnion

4.4 OPTION 2B

Option 2B is similar to Option 2A, but with additional construction staging for the concrete repair work. As previously noted, it was assumed that all construction stages for this option will be performed during a complete navigational outage with the bridge locked in the closed position. As such, an interim balance condition between construction staging is not required at this point. At completion, the total amount of replaced concrete for Option 2B would be the same as Option 2A. As a result, the findings in Option 2A also apply to Option 2B.

5 CONCLUSIONS

Based on the structural analysis of the Through truss, Counterweight truss, and Embedded truss members and the span balancing of all three (3) options, WSP is recommending that Option 2a – Partial Removal/Replacement of Concrete be further developed in Stage 2 of the Counterweight Rehabilitation Study. At this stage in the design process, the results dictate that Option 2a is currently the most feasible and does not required further investigation into the structural analysis, potential load postings on the structure during construction, or significant changes to the Counterweight design to maintain the span balance.

6 NEXT STEPS

To further develop the recommended rehabilitation strategy, the following recommendations will be considered given the overall condition of the Counterweight concrete and embedded steel truss.

6.1 REINFORCED CONCRETE

6.1.1 CONNECTING NEW CONCRETE TO EXISTING CONCRETE

Given the age and condition of the concrete remaining, the surface preparation of the concrete should be carefully considered. WSP recommends the surface of the existing concrete be damp for a period of at least 24 hours prior to concrete placement

In additional to providing an adequate bonding surface, the installation of steel dowels to a minimum depth of approximately 1000 mm beyond the removals is recommended given the overall depth of concrete to be reinstated. Further analysis in the detailed design phase would be required to determine the required spacing, edge distance, size, etc. of the dowels.

6.1.2 CLASS OF CONCRETE

Due to the harsh conditions the Counterweight is continuously exposed to; wind, rain, freeze/thaw, etc., it is recommended that a CSA A23.1-19 Class C1 concrete be used in future placements. This type of concrete will provide adequate strength and durability.

6.1.3 GROUTING OF CRACKS

Given the presence of cracked concrete throughout the Counterweight and especially in the openings, WSP is also recommending to epoxy inject cracks that remain after removal of concrete, specifically on the top and east faces, where voids are present. Additionally, all areas with excessive cracking should be injected as well. By sealing the cracks within the Counterweight, the internal structural steel is further protected in addition to maintaining the structural integrity of the existing concrete that will remain.

6.1.4 REINFORCING STEEL

WSP recommends the use of a premium reinforcing steel for the new concrete to be placed. Stainless Steel reinforcing is an ideal material because it is highly corrosion resistant. Additionally, since there are corrosion concerns with regular black reinforcing, Stainless Steel bars are chemically inert when placed adjacent to black reinforcing bars.

6.2 STEEL TRUSS

6.2.1 GIRDER F

Considering the Embedded truss members all have a D/C > 1 when applying a 50% section loss for Girder F members and a 5% section loss for all remaining members, rehabilitation of Girder F members are not required at this time.

WSP recommends that when the concrete is removed, those members are blasted to a near white condition, primed, and then coated for their protection. Further details will be determined during the detailed design phase.

6.2.2 STEEL PLATES

In addition to blasting and coating Girder F members, the removal of 600 mm of concrete for the North and South faces would present an opportunity to also blast and coat the inside of the two (2) metals plates on their respective faces.

6.3 GALVANIC ANODE

With the presence of chlorides within the Counterweight concrete, WSP recommends the use of a galvanic anode system to extend the service life of the reinforced concrete. Further investigation and design of a galvanic anode system will be determined during the detailed design phase.

6.4 CONSTRUCTION

6.4.1 CRASH DECK SYSTEM

To effectively and efficiently remove and replace the Counterweight Concrete, WSP recommends the use of a crash deck falsework/formwork system. During removal of the concrete, the crash deck would prevent any concrete from falling to the roadway surface below, protecting vehicles and pedestrians. Additionally, once all the removals are completed, the falsework system would act as a working platform during reconstruction and finally once concrete is placed, the system could act as formwork. While this option may be expensive, it would allow at least one of traffic at all times during construction.

The crash deck falsework/formwork system would be further developed in Stage 2 of the Counterweight rehabilitation Study in accordance for the Terms of Reference.

6.4.2 ACCESS HATCHES

Given that the steel members of Girder F have been constantly exposed to the elements and moisture, WSP recommends that new, watertight access hatches be placed on the top face of the Counterweight. Further details will be developed during the detailed design phase.

In addition to the access hatches on the top face, WSP recommends the access hatches/doors on the East face of the Counterweight be removed and replaced. During the site investigation it was noted these access hatches/doors needed replacement. Currently WSP recommends the door be replaced in kind, however, their design will be further investigated in the detailed design phase.

Trunnion construction counts on counterweight (likely) and must be considered. Imparts load at the top joint, pulling their truss up