LASALLE CAUSEWAY BASCULE BRIDGE

COUNTERWEIGHT REHABILITATION STUDY

Inspection Report | FINAL DRAFT

PSPC Project No. R.108246.001

March 30, 2021

PREPARED FOR:

Public Works and Government Services Canada

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	Dec 18, 2020	Draft Report	CC/TM	JB	MM	
2	Jan 22, 2021	Final Draft	CC/TM	JB	MM	
3	Mar. 30, 2021	Final	TM	JB	MM	

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EXECUTIVE SUMMARY

The Lasalle Causeway is located in Kingston, Ontario and was opened to the public in 1917. The existing 550 tonne concrete counterweight was part of the original construction and is suspended from the counterweight truss above the roadway. No known repairs have been conducted to the Counterweight in the past. The purpose of this detailed field investigation was to determine the condition of the concrete Counterweight to facilitate the development of short term and long-term rehabilitation options. The scope of the field investigation included concrete condition survey, concrete coring, and condition assessment of the embedded Counterweight steel truss.

There is a steel truss interior to the Counterweight constructed of built up sections of plates, channels, angles, and lattice embedded in the Counterweight concrete. The Counterweight is also reinforced at the exterior faces by steel reinforcing bars and wire mesh, with the steel plates on the North and South faces of the Counterweight being secured in place by threaded steel rods. All faces except the North and South face are covered with what appears to be corrugated metal roof panels, though the exact material and installation date are unknown.

The LaSalle Bascule Bridge was closed to vehicular traffic for three days from October 10, 2020 at 0:00 hours to October 13, 2020 at 6:00 hours to facilitate a safe inspection of the Counterweight. The Counterweight study was conducted by Sub-contractor Haghbin & Associates Ltd (HAL) under the direct supervision of WSP. Sections of panels were removed to facilitate visual and hands-on inspection, which was documented on drawings WSP prepared prior to inspection for use by HAL.

To ensure the structural steel members within the Counterweight were not damaged during coring operations, WSP prepared a 3D model of the structural steel embedded in the Counterweight, as detailed in the original contract drawings. WSP analyzed the location of the structural steel members using the 3D model to determine the optimal locations for coring.

The Counterweight concrete is overall in very poor condition, showing severe signs of freeze-thaw damage and reinforcing corrosion, which has resulted in severe delamination, spalling, and cracking. This severe damage was measured to extend to an approximate depth 300 mm in some locations, as measurements were taken at the North West corner on the bottom face of the Counterweight. Concrete durability issues are not only present on the surface, but also within the Counterweight. Exposed reinforcing mesh and large cracks were observed on the West face. The top face was covered in roofing tar and unable to be visually inspected but hammer sounding and coring revealed severe delamination and significant cracking at a depth from the surface. Concrete was however inspected inside each of the four compartments on the West side of the top face, with metal debris noted, and concrete in very poor condition showing excessive cracking at the base of each compartment. Cladding was not removed from the entire South East face but the observed condition in the two sections removed was similar to the West face. Two compartments exist on the East face, with steel doors that are in poor condition and were difficult to open and close. Interior concrete is in poor condition showing map cracking and efflorescence on the ceiling of each compartment and areas of surface scaling on the walls. The bottom face was similarly observed to be in poor condition. The entire surface and to some depth of the concrete is deteriorated and particularly in the bottom 600 mm there are areas where the deterioration appears more advanced.

A total of seven cores, two of which were greater than half the counterweight depth, were extracted from the West, East and top faces. After the cores were extracted, the internal concrete condition at the locations of the cores was visually documented using a fibre scope and eextracted cores were tested for compressive strength. Overall, cores removed from all faces are in fair to poor condition showing excessive cracking in the first 1,300 mm, after which sound concrete is reached. Concrete close to the surface is shown to have a compressive strength of only 10.4 MPa, concrete at a depth of 750 mm – 1600 mm is shown to have strengths in the range of 12.3 MPa – 14.7 MPa after which strength reaches almost 20 MPa at the depth of beyond 1850 mm. The carbonation depth on the West face is in the range of 90 mm – 115 mm. A petrographic examination determined concrete in the counterweight was overall determined to be poorly proportioned and non-air entrained, but moderately well consolidated. The water to cement ratio of the concrete is estimated to be more than 0.5.

The condition of the Counterweight steel truss was assessed through limited concrete removals and the available access hatches from the top of the Counterweight. Located approximately 300 mm within the core, a small portion of steel plate were exposed at the South East and North East corners through coring of the concrete and showed very little to no deterioration and were in good condition. Exposed structural steel within the open hatches was visible and local section loss of 30 to 50% was visible. Limited concrete removals were completed at the North West corner of the top face of the counterweight by chipping and steel near the upper edge of the concrete showed surface rusting with some limited section loss. Much of this steel was observed during the 2003 Fatigue repairs when the connections were exposed. Based on these observations it appears that the concrete can protect the steel but that when not cast in the concrete or when near the surface of the concrete surface rust should be expected and the implications should be assessed.

Without the cladding small to medium pieces of concrete would likely regularly fall to the roadway. Concern was expressed regarding the need for an interim support and this is being addressed in a separate program. The information gathered and presented in this current condition report will feed into the structural assessment and rehabilitation design.

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 detailed field investigation was to determine the condition of the concrete Counterweight to facilitate the development of the structural analysis and conceptual design of short term and long term rehabilitation options.

The goal of the investigation was to confirm the overall condition of the Counterweight, capture elements in need of repair or replacement, and determine if any emergency repairs are required. Additionally, missing information was to be captured, such as missing dimensions not on existing drawings. Analysis will follow to establish limits for additional weight to the Counterweight structure and determine rehabilitation needs and timelines.

1.3 SCOPE

The scope of the field investigation was to determine the condition of the Counterweight. This included:

- Concrete condition survey
  - Hammer sounding
  - Deterioration mapping
- Concrete coring

  - Fiber scope of inside of cores
  - Concrete carbonation depth test
  - Material testing
    - Compression strength testing
    - Carbonation testing
    - Petrographic analysis
- Condition assessment of the embedded Counterweight steel truss

This inspection memorandum includes the review and analysis of field and lab results. A description of the structure, the methodology, and the finding of the Counterweight inspection are detailed in the remainder of this report. As per the Terms of Reference (ToR), a separate report will be prepared that includes comments on structural integrity of the Counterweight and potential short-term and long-term rehabilitation options.

2 STRUCTURE DESCRIPTION

The roadway truss portion of the bridge measures 48.8 m (160’-0”) from toe end panel point to the main trunnion, an additional 12.8 m (42’-0”) from the main trunnion to the Counterweight trunnion, and another 10.4 m (34’-1”) to the end of the concrete Counterweight when the bridge is in the bridge-lowered position. These dimensions are shown in **Figure 2-1** below. The width of the bascule span is 8.2 m (27’-0”) from centerline to centerline of the bascule trusses. There is an additional 1.3 m (4’-3”) at the sidewalk level extending from the South bascule truss at its panel points for the sidewalk.

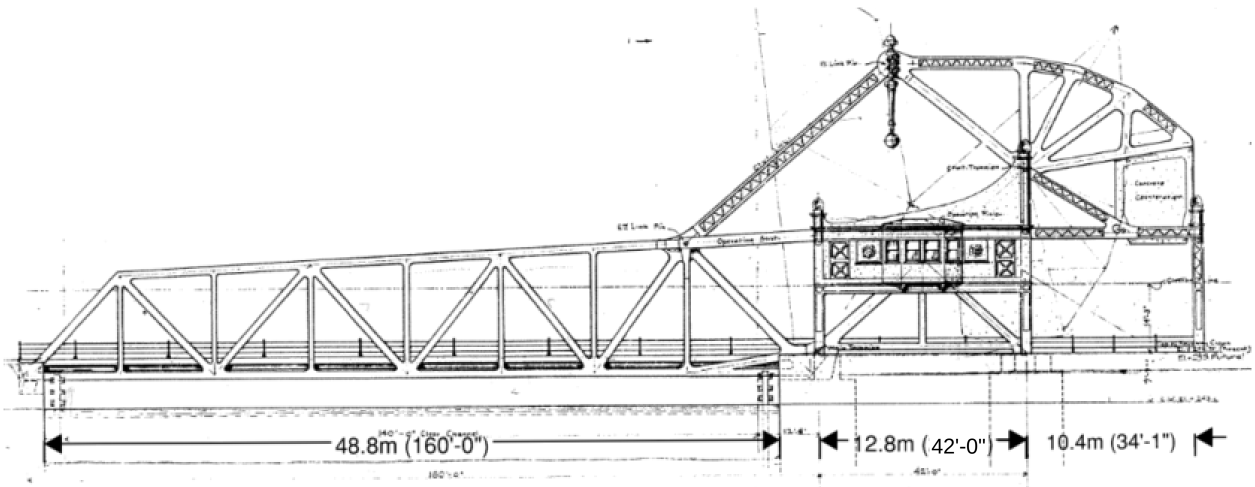


Figure 2-1: Bascule portion of bridge dimensioned

The concrete Counterweight is understood to be from the original construction of the bridge in 1917, with no known repairs completed since. The Counterweight is suspended from the Counterweight truss above the roadway and weighs approximately 550 tons (1,220,000 lbs). There is a steel truss interior to the Counterweight constructed of built up sections of plates, channels, angles, and lattice embedded in the Counterweight concrete. Refer to **Appendix D** for a 3D model of the Counterweight truss. The Counterweight is also reinforced at the exterior faces by steel bars and wire mesh, with the steel plates on the North and South faces of the Counterweight being secured in place by threaded steel rods. All faces except the North and South face are covered with what appears to be corrugated metal roof panels, though the exact material and installation date are unknown. These panels, though providing some protection from the elements, are not watertight and therefore hinder proper drying of the concrete once it gets wet. The panel supports and panels are known to be corroding, primarily on the underside, and there are several small perforations on the East face. Two chambers are provided in the East face of the Counterweight to allow for the addition of dead load as required to balance the bridge.

3 INSPECTION METHODOLOGY

The LaSalle Bascule Bridge was closed to vehicular traffic for three days from October 10, 2020 at 0:00 hours to October 13, 2020 at 6:00 hours (Thanksgiving Weekend) to facilitate a safe inspection of the Counterweight. The bridge remained accessible to pedestrian and cyclist traffic as well as emergency vehicles for the duration of the closure, as the sidewalk was kept clear and an inspection team member was available to safely guide pedestrians/cyclists through the inspection area. The bridge was not lifted during the inspection, though boat traffic was invited to book an appointment for lifting, had it been required. The Counterweight study was conducted by Sub-contractor Haghbin & Associates Ltd (HAL) under direct supervisor of WSP.

All debris that fell off or was removed from the Counterweight was collected and weighed by PSPC. This will be used when checking the balance of the bridge.

3.1 CONCRETE CONDITION SURVEY

To conduct the concrete condition survey, HAL utilized a 60-foot Genie boom lift and Scissor Lift to access the side faces of the Counterweight to remove (and re-instate prior to opening the bridge to vehicular traffic) the following sections of the Cladding, as shown in **Figure 3-1**:

— Entire West face of the Counterweight;
— Entire top face of the Counterweight;
— Two large sections of panels on the bottom face, one on the North and one on the South side; and
— Full height section of panel on the South side of the East face.

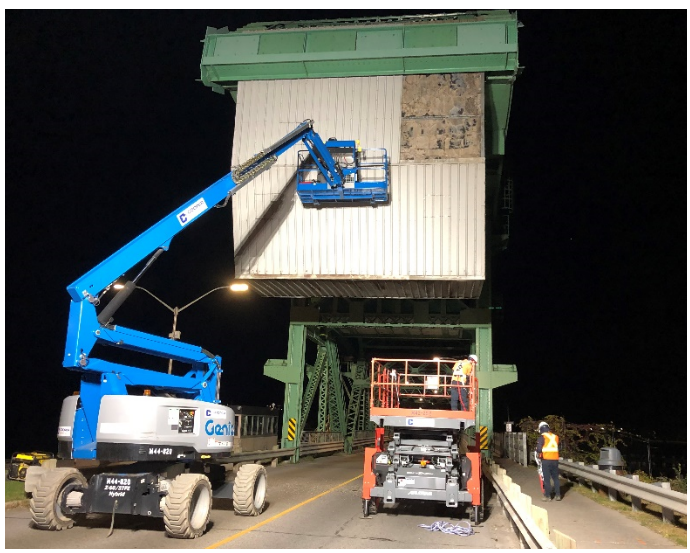


Figure 3-1: Removal of Counterweight cladding. West face shown.

Once the sections of panels were removed, HAL conducted a condition survey of the exposed concrete by means of visual and hands-on inspection methods. The hands-on inspection utilized hand tools, such as hammers, to conduct hammer sounding of the concrete to determine the extent of spalls and delamination. WSP prepared drawings prior to inspection for use by HAL in documenting all results from the visual and hands-on inspection, which can be found in **Appendix C.** Additionally, the two voids on the East face and all four hatches on top of the Counterweight were inspected.

HAL was able to locally remove up to 300 mm of the concrete at the NorthWest bottom corner of the Counterweight using hand tools to determine the depth of deterioration.

3.2 CONCRETE CORING

To ensure the structural steel members within the Counterweight were not damaged during coring operations, WSP prepared a 3D model of the structural steel embedded in the Counterweight, as detailed in the original contract drawings, shown in **Figure 3-2**. Additional views of the model can be seen in **Appendix D**.

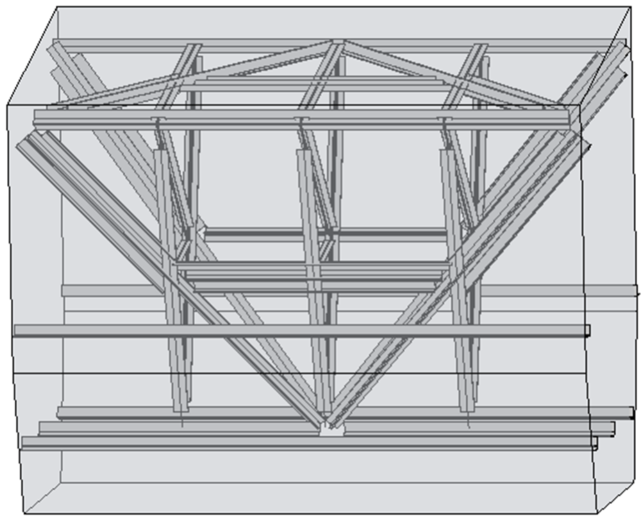


Figure 3-2: 3D model of embedded Counterweight steel – isometric view from East

WSP analyzed the location of the structural steel members using the 3D model to determine the optimal locations for coring. These locations were marked on the model and this information was provided to HAL in order to assist them while coring. Additionally, the Counterweight is reinforced at the exterior faces by steel reinforcing bars and wire mesh. Ground Penetrating Radar (GPR) was used in an effort to locate the reinforcing steel and wire mesh, however the small cell spacing of the wire mesh prevented the signal from penetrating the mesh to properly identify the reinforcing steel.

A Hilti coring machine was used to extract the nominal 100 mm diameter cores.

Figure 3-3: Hilti coring machine mounted on Counterweight (core location C1 shown)

Cores were removed at the following locations:

- One full depth core (core C2) approximately 2,600 mm in length was removed, extending from the West face to the inside of a chamber on the East face.
- A core (core C1) measuring approximately 2,700 mm was removed from the top right corner of the West face.
- Three additional horizontal cores (cores C3, C4 and C5) ranging from 300 mm to 600 mm were removed at various locations on the Counterweight to check the depth of delamination and condition of concrete at a depth.
- Two cores (cores C6 and C7) were also removed from the top face at both NorthEast and SouthEast corners, respectively.

A core log, along with core drawings and locations is presented in **Appendix B**.

After the cores were extracted, the internal concrete condition at the locations of the cores was visually documented using a fibre scope. A concrete carbonation depth test was conducted at select locations. All concrete core holes were patched with Sika 45 prior to reinstatement of the cladding panels.

Minimal reinforcement was encountered during coring operations, however, a small sample of reinforcing was removed with core C1 and will be used to determine the reinforcing condition on the interior of the Counterweight. Concrete cores will be tested for compressive strength and petrographic analysis.

3.3 COUNTERWEIGHT STEEL TRUSS

To access the top of the Counterweight, a rope access team was utilized to set up tie-off points for the WSP and HAL inspectors. The Counterweight was reached via permanent stairs and catwalk.

Figure 3-4: Rope access on top of Counterweight

Upon removal of the top face cladding panels, the top of the Counterweight was assessed, and it was determined that fresh concrete/grout had been poured on the NorthWest and SouthWest corners in 2003 to allow the replacement of rivets. Concrete cores were extracted at the NorthEast and SouthEast corners on the top of the Counterweight in order to expose a structural steel plate and document its condition in these locations. It was anticipated that the condition of steel at these locations would be indicative of the infiltration of water and chlorides (i.e. road salts) and could be used to conclude the condition of structural steel further embedded in the Counterweight.

A chipping hammer was used to form a test pit to expose structural steel on the NorthWest corner at the top of the Counterweight. This area was previously exposed during the rivet replacement. The current structural steel condition was noted to compare with previously documented condition. The condition of the exposed structural steel within the access hatches at the top of the Counterweight was also reviewed and documented in **Section 4.3.**

4 INSPECTION RESULTS

4.1 CONCRETE CONDITION SURVEY

The concrete Counterweight is covered on all sides, except North and South faces, by steel roofing cladding that is attached to the concrete Counterweight using angle iron anchored to the Counterweight. Overall, the cladding is in fair to poor condition showing signs of surface staining and localized corrosion, which is more pronounced on the bottom face, which is likely where moisture accumulates. The angle iron anchored to the Counterweight is also in fair to poor condition showing localized areas of severe corrosion. Angle iron is mounted firmly to the concrete except for the bottom right hand corner of the West face, which was replaced during the condition assessment. Upon removal of steel cladding from the top face, the concrete is covered with what is believed to be a roofing tar material. The purpose and timing of application of this material is unknown, but most likely it was applied to mitigate the penetration of moisture.

The underlying concrete is overall in very poor condition, showing severe signs of freeze-thaw damage and reinforcing corrosion, which has resulted in severe delamination, spalling, cracking and section loss. Concrete durability issues are not only present on the surface, but also within the Counterweight as further described in **Section 4.2**.

This section briefly defines the condition of the concrete Counterweight. Additional pictures of each face are presented in **Appendix A** (Figures 1-25).

4.1.1 WEST FACE

Overall, the condition of the West face is in poor condition showing areas of spalling, delamination, and cracking as presented in **Figure 4-1** (see additional photographs (Figures 1-25) in **Appendix A**) Exposed reinforcing mesh is found in the middle portion of this face where approximately 50 mm of concrete has been lost due to spalling. The remainder of the face appears to be original concrete, however, is delaminated (as confirmed by hammer sound testing) and can easily be removed with minimal effort.

The area of primary concern is at the very top of the West face where large cracks (i.e. approximately 15-25 mm in width) and spalling are present, as illustrated in **Figure 4-2**. While it was not possible to measure the overall depth of the crack, it may be connected to cracking found inside the four compartments on the top face (**Figure 4-6**). The reason for this cracking cannot easily be determined by visual review; however, based on the dimensions of the structure (which can be classified as mass concrete) and the exposure condition, thermal-induced cracking, shrinkage cracking and cyclic freezing and thawing may have contributed to the observed cracking.

Three cores (C1, C2 and C5) were removed from this face (top middle, middle right (South), and bottom left (North)). These cores and core holes, were used to measure compressive strength and carbonation depth as presented in **Section 4.2.1** and **4.2.2** respectively.

Figure 4-1: West face of Counterweight showing signs of spalling, delamination, and reinforcing corrosion (approximate core locations and areas of spalling and delamination are presented in Appendix C, detailed core locations found in Appendix A)

Figure 4-2: Crack (15 mm – 25 mm in width) running along top edge of West face.

4.1.2 TOP FACE

Upon removing steel cladding from the top face, it was observed that the concrete is covered with what appears to be roofing material/tar as presented in **Figure 4-3**.

While the underlying concrete was unable to be visually examined, hammer sound testing confirmed the presence of delamination of the underlying concrete. Areas of delamination were also confirmed following the removal of cores (C6 and C7) from both SouthEast and NorthEast corners of the top face where cracking parallel to the surface of the top face was found at depths of 50 mm, 90 mm, 110 mm, 180 mm and 300 mm from the surface (see Core Logs in **Appendix B**).

Four individual compartments are also found on the top face as presented in **Figure 4-3**. The internal dimensions of the compartments are not know given the amount of debris inside and their varying size. Concrete was inspected inside each of these compartments, with metal debris noted, as presented in **Figure 4-5**. The metal debris has been noted during previous inspections of the Counterweight and may be removed during a long-term rehabilitation as the reason for its presence is unknown at this time.

Figure 4-3: Top face covered with roofing material. North half of top face shown.

Figure 4-4: Top face covered with roofing material. South half of top face shown.

Figure 4-5: Metal debris inside compartment on top face of Counterweight.

Overall, the concrete condition inside the four compartments is in very poor condition showing excessive cracking at the base of each compartment along with what appears to be construction joints. Fragments of concrete were also found at the base of each compartment, which are likely the result of freeze-thaw damage causing spalling as presented in **Figure 4-6**. Cracking at the base of these compartments are a concern as they may be connected to the large crack at the top of the West face. While outside the scope of this report, measures to address concrete cracking will be submitted in a separate report.

Figure 4-6: Fragments of deteriorated concrete and cracking along construction joint inside compartment on top face (second from South end) of Counterweight.

4.1.3 EAST FACE

Two vertical panels (full height) were removed from the South end of the East face. Cladding was not removed from the entire face as strong conclusions as to the concrete condition were already established and it was determined that removing additional panels was unlikely to reveal further information. This also ensured that the bridge would be ready to safely reopen as scheduled. As observed on the West face, the underlying concrete is in poor condition showing areas of spalling, delamination, cracking and section loss as presented in **Figure 4-7, Figure 4-9,** and **Figure 4-11**. Exposed reinforcing steel showing generalized corrosion is present on the bottom corners of the East face where approximately 120 mm – 300 mm of concrete has been lost (see **Figure 4-9** and **Figure 4-11**).

Figure 4-7: Bottom South East corner of East face showing cracking and areas of spalling. South end of bottom face also shown.

Figure 4-8: Zone indicated by red box in Figure 4-7 (Bottom South East corner of East face).

Figure 4-9: Concrete section loss and exposed reinforcing steel showing generalized corrosion of South end of bottom face.

Figure 4-10: Zone indicated by red box in Figure 4-9 (South East corner of bottom South face).

Figure 4-11: Concrete section loss at bottom North East bottom corner of East face.

Figure 4-12: Zone indicated by red box in Figure 4-11 (North East bottom corner of East face).

Similar to the top face, the East face has two compartments, with steel doors as shown in **Figure 4-13** and **Figure 4-14.** Overall, the steel doors are in poor condition and were difficult to open and close. Concrete inside each compartment found on the East face is also in poor condition showing map cracking and efflorescence on the ceiling of each compartment and areas of surface scaling on the walls. In an attempt to determine the performance of sound concrete, not directly exposed to cyclic freezing and thawing, cores C3 and C4 were removed from inside of the right compartment.

Figure 4-13: Workers inspecting South compartment on East face

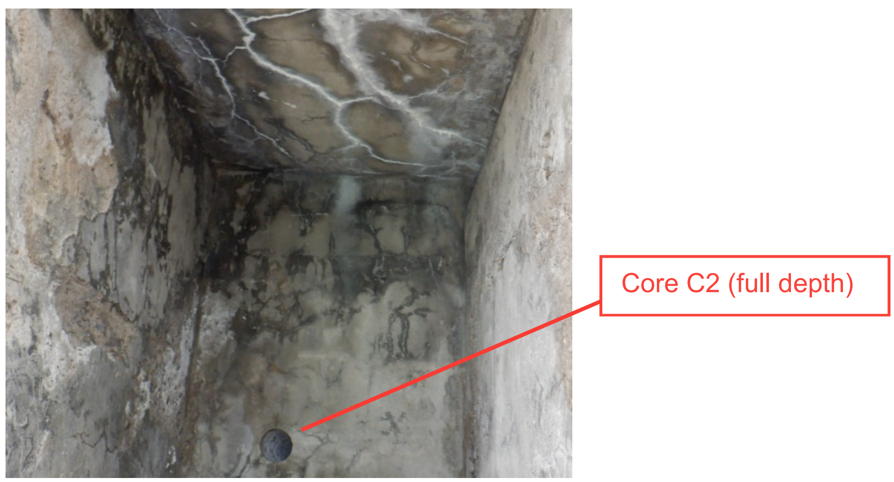


Figure 4-14: Ceiling of South compartment showing sever signs of map cracking and efflorescence. Full depth (core hole C2) shown.

4.1.4 BOTTOM FACE

Two panels were also removed from the bottom face at the North and South ends. Overall, the underlying concrete is in a similar condition to the rest of Counterweight showing cracking, spalling, and delamination reaching depths in the range of 100 mm – 200 mm. To determine the condition of underlying reinforcement, a test pit 300 mm in depth was performed on the NorthEast corner of the bottom face. Reinforcing steel and mesh are in poor condition showing generalized corrosion on all faces in addition to localized areas of corrosion where approximately 25% of the diameter has been lost. Visible concrete loss was also visible in all four corners of the bottom face.

Figure 4-15: Bottom face showing panels removed from North and South ends

Figure 4-16: Condition of underside of counterweight. North end shown

Figure 4-17: Condition of underside of counterweight. South end shown.

4.2 CONCRETE CORING

A total of seven (7), nominal 100 mm diameter, cores were removed from the Counterweight at predetermined locations to avoid structural steel. Core locations and lengths are presented in **Table 4-1**. Photographs of core locations are presented in Figure 4.1 (West face – core C1, C2 and C5), Figure A27 (East face – core C3 and C5), Figure A20 and A21 (top face – core C6 and C7) in **Appendix B**. Due to the depth of delamination on the West face, approximately 50 mm – 100 mm of concrete was first removed to ensure the coring machine was mounted to sound concrete.

Table 4-1: Concrete Core Details

CORE NUMBER[1]	CORE LOG IDENTIFICATION (APPENDIX B)[2]	FACE	LOCATION ON FACE	APPROXIMATE LENGTH (MM)
C1	Core 1A-D	West Face	Top Middle	2,650
C2	Core 2A	West Face	Middle Right	2,600
C3	Core 3	East Face	Right Compartment	480
C4	Core 4	East Face	Right Compartment	720
C5	Core 5	West Face	Bottom Left	395
C6	Core 6	Top Face	South East Corner	220
C7	Core 7	Top Face	North East Corner	335

Note:
1. WSP core log reference nomenclature
2. HAL core log reference nomenclature

Overall, cores removed from all faces are in fair to poor condition showing excessive cracking in the first 1,300 mm, after which sound concrete is reached. Both angular and round aggregate appear in all cores ranging in diameter from 25 mm to 100 mm. Minerology, source and other aggregate properties are presented in **Section 4.2.3** in addition the full petrography report found in **Appendix E**.

Cores removed from the West face (C1, C2, and C5) show excessive cracking throughout their lengths with areas of sound concrete at depths of 1,100 mm – 2,030 mm in core C1 and 1,330 mm – 2,400 mm in core C2, respectively. Concrete in core C1 at a depth of approximately 2,030 mm until the end of the core is in very poor condition, which may be at a location and elevation near the bottom of one of the compartments found on the top face. Upon removing core C2, steel plate was hit at depths of 1,000 mm and 1,300 mm. While this is not structural steel, area void is found between these plates (distance of approximately 300 mm) which is partially filled with loose angular aggregate with no bonded cement paste. The significance of this void is unknown and is filled with aggregate different than that used in the surrounding concrete. Core C2 also penetrated the entire width of the Counterweight from the West face to inside the South (left) compartment on the East face (see Figure A29). Compressive strength results tested on sections from both core C1 and C2 are presented in **Section 4.2.1**.

Cores removed from inside the North (right) compartment on the East face (C3 and C4) are in similar condition showing excessive cracking along their lengths. Both cores were removed from inside the right compartment on the East face in an attempt to collect samples from as close to the center of the Counterweight as possible.

Finally, cores C6 and C7 were removed from the top face (see Figure A20 and A21). Horizontal cracks at depths of 50 mm, 90 mm, 130 mm and 240 mm represent areas of delamination down to where structural steel was reached. Compressive strength testing was unable to be performed on these sections due to the presence of excessive cracking.

4.2.1 COMPRESSIVE STRENGTH

Compressive strength was determined on section of cores as presented in **Table 4-2**. Sections of cores were selected to determine compressive strength relative to depth. Specimens were prepared in accordance with CSA A23.2-14C *Obtaining and testing drilled cores for compressive strength testing* and tested for compressive strength in accordance with CSA A23.2-9C *Compressive strength of cylindrical concrete specimens.* Selected samples are presented in Table 2 in addition to compressive strength results.

Table 4-2: Compressive Strength Results

CORE NUMBER	FACE	SAMPLE ID	DEPTH FROM FACE(MM)	COMPRESSIVE STRENGTH (MPA)
C1	West	C1-1	300-535	10.4
		C1-2	750-950	14.7
		C1-3	1300-1500	14.3
C2	West	C2-1	1400-1600	12.3
		C2-2	1850-2100	19.8
C4	East	C4-1	40-250	16.6
			Average	**14.70**
			St. Dev	**3.28**

As expected, compressive strength results in cores C1 and C2 removed from the West face show an increase in compressive strength relative to depth. Concrete close to the surface is shown to have a compressive strength of only 10.4 MPa which is generally in line with the compressive tests of cores taken by Parsons in 2018 (compressive strength of 11.9 MPa at a depth of 50 mm – 250 mm). Concrete at a depth of 750 mm – 1,600 mm is shown to have strengths in the range of 12.3 MPa – 14.7 MPa after which strength reaches 19.8 MPa at the depth between 1,850 mm – 2,100 mm. While core location C2-1 is at a depth similar to that of C1-3, the lower strength may be the result of hairline cracking observed along the length of core at this location. In accordance with the MTO Structural Rehabilitation Manual (SRM) concrete with compressive strengths less than 20 MPa is considered of poor quality.

The reduced strength closer to the surface (outer 1,600 mm) of the Counterweight is assumed to be the result of exposure to cyclic freezing and thawing, and possibly thermal cracking during concrete hydration. Concrete has been known to reduce compressive strength to 60% of its original strength due to exposure to cyclic freezing and thawing[1].

4.2.2 DEPTH OF CARBONATION

Carbonation depth was measured on site using phenolphthalein indicator solution, which in an acid-based indicator solution. Phenolphthalein is a convenient means of measuring depth of carbonation as it changes from purple (pH > 9.2) to colourless (pH < 9.2). A pH level less than 9.2 is a concern as steel reinforcement is not capable of maintaining its ferric oxide passive layer, which will lead to corrosion in the presence of moisture and oxygen.

Phenolphthalein was sprayed on freshly fractured concrete as well as in core holes to determine the depth of carbonation as presented in **Figure 4-18**. Carbonation depths of approximately 40 mm were found in core holes on the West face. Assuming 50 mm – 75 mm of concrete was removed at these locations to ensure the coring machine

[1] Shang, H.S., Cao, W.Q. and Wang, B. (2014). Effect of Fast Freeze-Thaw Cycles on Mechanical Properties of Orginary-Air-Entrained Concrete. The Scientific World Journal. Vol 14

was mounted to sound concrete, it can be assumed that the carbonation depth on the West face is in the range of 90 mm – 115 mm. Based on original drawings and steel reinforcements penetrated during coring, the carbonation depth has reached reinforcing mesh and a layer of reinforcement at cover depths of 50 mm and 55 mm, respectively.

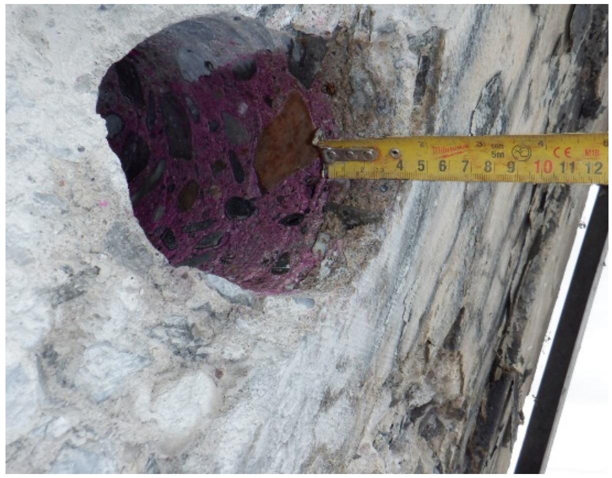


Figure 4-18: Carbonation depth at core location C2

Carbonation occurs at an optimum relative humidity of 45% - 65%, which may be present under the steel cladding placed on the exterior of the Counterweight. Carbonation is a concern because, (i) corrosion of steel may be initiated once the carbonation front reaches the surface of the steel; and (ii) carbonation may render the near-surface concrete less resistance to abrasion, salt scaling and chloride ingress.

4.2.3 PETROGRAPHIC EXAMINATION

A petrographic examination in accordance with ASTM C856/856M-20 "Standard Practice for Petrographic Examination of Hardened Concrete" was performed on select sections of cores C1 and C2 to examine the condition of the concrete relative to the depth. Sections from cores C1 and C2 were analyzed at depths of 1,550 mm – 1,750 mm (core C1) and 0 mm – 200mm (core C2). The detailed petrographic examination report in presented in **Appendix E**.

According to the report, concrete in the counterweight was overall determined to be poorly proportioned and non-air entrained, but moderately well consolidated. The water to cement ratio of the concrete is estimated to be more than 0.5. Hardened air void analysis was previously performed by Parsons in 2018. Air content, spacing factor and specific surface were determined to be 2.25, 0.545mm and 12.56mm^2/mm^3, respectively[2]. In accordance with CSA A23.1 Clause 4.3.3.3, concrete requires air contents in excess of 3, spacing factors less than 0.2mm, and recommends a specific surface greater than 24 mm^2/mm^3.

Deterioration was quantified in accordance with the Damage Rating Index (DRI) as described in the report. Concrete at the surface had a DRI value of 440 which indicates considerable cracking and damage whereas concrete at depth had a DRI value of approximately 50 indicating concrete in generally good condition. Deterioration at the surface is due in part to alkali-aggregate reaction but freezing and thawing of non air-entrained concrete also played a significant role. Both coarse and fine aggregate were assumed to be from the same source, however, found to vary in composition and proportions. Coarse aggregate particles were determined to be sound, un-weathered and suitable for concrete aggregate; however, localized aggregates contained small dolomite rhombohedra and reactive siliceous minerals indicative of alkali-aggregate reaction, which is likely the cause of map cracking on the ceilings of compartments on the East face.

[2] Parsons (2019). LaSalle Causeway Bascule Bridge 2018 Comprehensive Detailed Inspection Report

Additional details are found in the petrographic examination presented in **Appendix E**.

4.3 COUNTERWEIGHT STEEL TRUSS

The condition of the Counterweight steel truss was assessed through limited removals and the available access holes from the top of the Counterweight. It was noted upon removing the cladding that a black coating resembling roof tar had previously been placed on the concrete face, presumably to act as waterproofing. Core holes (cores C6 and C7) to allow inspection on the North East and South East corners of the top of the counterweight concrete were drilled, revealing two horizontal gusset plates approximately 300 mm deep, as illustrated in **Figure 4-19**. The embedded steel plates showed very little deterioration. It is inferred given the condition of the plates that the steel further into the Counterweight would likely also be in good condition, as the potential infiltration of water should be less severe with depth.

Figure 4-19: Counterweight truss steel plate exposed. Core hole C7 shown.

However, there are four hatches opening to voids on the West side of the top of the Counterweight. The hatches were opened to reveal exposed structural steel, and it was noted that the exposed member ('Girder F' on the original contract drawings) showed severe deterioration. It has been exposed to atmosphere and moisture (within one of the center voids) and is showing local section loss of 30% - 50%. Depending on the outcome of structural evaluation, it may require local steel rehabilitation.

Finally, a concrete test pit was removed from the NorthWest corner to reveal steel that was previously exposed during the 2003 fatigue connection retrofit to see if there was significant loss of section. The area had surface rust but no significant loss of section which was similar to the previous inspection.

Additional information regarding the condition of the Counterweight steel truss is available from a fatigue inspection and repair completed in 2003. The 2003 fatigue repair required removal of concrete near the connections to allow rivet replacement. In addition, two panels were removed to look at the surface of the concrete. This and the more targeted Parsons inspection may be the only documented information regarding the concrete since the time the cladding was installed, which is believed to have occurred in the 1970s or 1980s. The condition of the steel at the connections which was exposed during the fatigue inspection is similar to that of the steel exposed during the current inspection, indicating that any steel deterioration has not progressed in a significant manner.

Based on these observations it appears that the concrete can protect the steel but that when not cast in the concrete or when near the surface of the concrete surface rust should be expected and the implications should be assessed.

5 CONCLUSION

In conclusion, closure of the LaSalle Bascule Bridge over Thanksgiving weekend allowed for a complete and detailed inspection of the Counterweight concrete and steel condition. Much of the cladding was removed to facilitate deterioration mapping of the concrete surface and concrete coring, and to allow for portions of the structural steel to be exposed.

The concrete faces were found to be in poor condition, showing significant delamination, spalling, and cracks and section loss. This severe damage extended to a depth of 100 mm - 300 mm in some locations. Extracted cores were tested for compressive strength, showing that the strength of the concrete increases with depth into the Counterweight, to a maximum observed compressive strength of 19.8 MPa, though the average of the four cores tested was 14.7 MPa. All concrete tested is below 20 MPa which the MTO Structural Rehabilitation Manual (SRM) considers of poor quality

The embedded structural steel, observed from the top face, was found to be in good to fair condition with little evidence of infiltration of water and chlorides. The exposed concrete within the hatches however was found to be in poor condition, due to exposure to atmospheric conditions and moisture.

The consistency of the results over the entire area of the counterweight supports the conclusion that sufficient information was gathered during the inspection. A separate report will be submitted outlining the structural assessment and evaluation as well as balancing calculations for the Counterweight. This additional report will also present the recommendations for short-term and long-term rehabilitation needs based on the findings of this inspection and subsequent analysis.

APPENDIX

A

SITE PHOTOGRAPHS

APPENDIX A

Figure A1 – Steel cladding removed from top half of West face. Underlying concrete showing areas of spalling, and map cracking.

Figure A2 – Delamination, surface cracking and exposed reinforcing mesh. Concrete was originally designed with a concrete cover depth of 50mm over the reinforcing mesh.

APPENDIX A

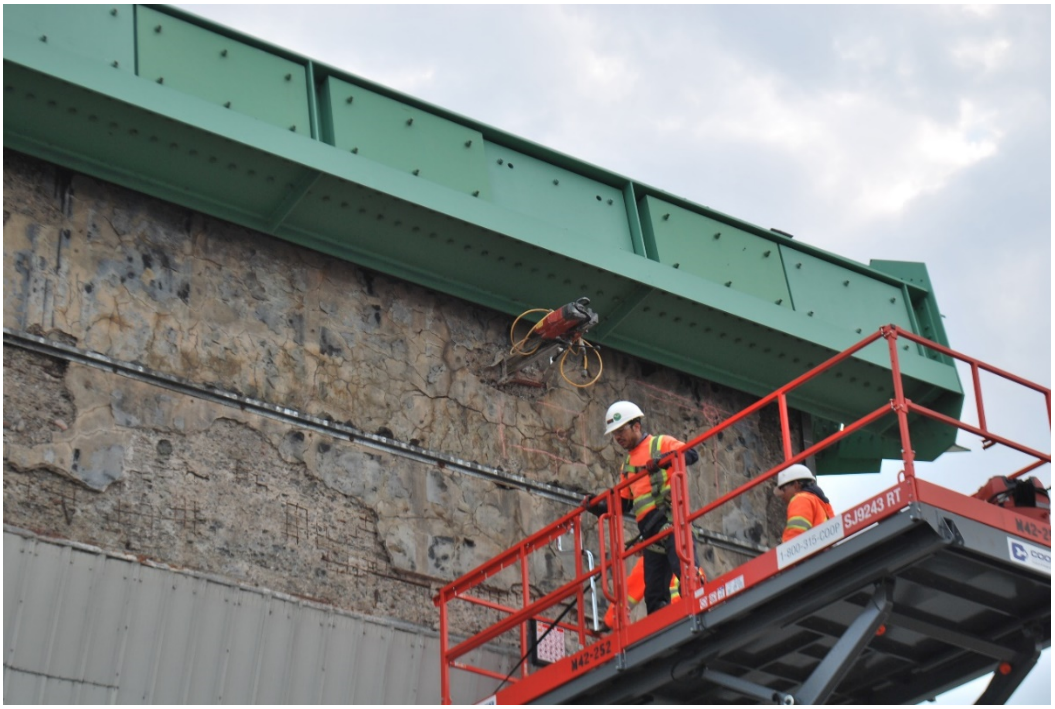


Figure A3 – Coring rig installed on West face at core location C1. A core approximately 2700mm in length was removed from this location.

Figure A4 – Concrete coring in progress at location C1.

APPENDIX A

Figure A5 – Concrete coring at location C1. Area surrounding core location shows original yet delaminated concrete. Exposed reinforcing mesh and spalled concrete is present.

Figure A6 – Steel cladding removed from entirety of West face. Spalling and exposed reinforcing mesh in center portion of the face in addition to areas of delamination and cracking at top and bottom areas.

APPENDIX A

Figure A7 – North West bottom corner of West face showing concrete loss, spalling, exposed reinforcing steel and surface cracking. Corrosion observed along angle iron embedded in concrete.

Figure A8 – South West bottom corner of West face showing concrete loss, spalling, exposed reinforcing steel and surface cracking. Corrosion observed along angle iron embedded in concrete.

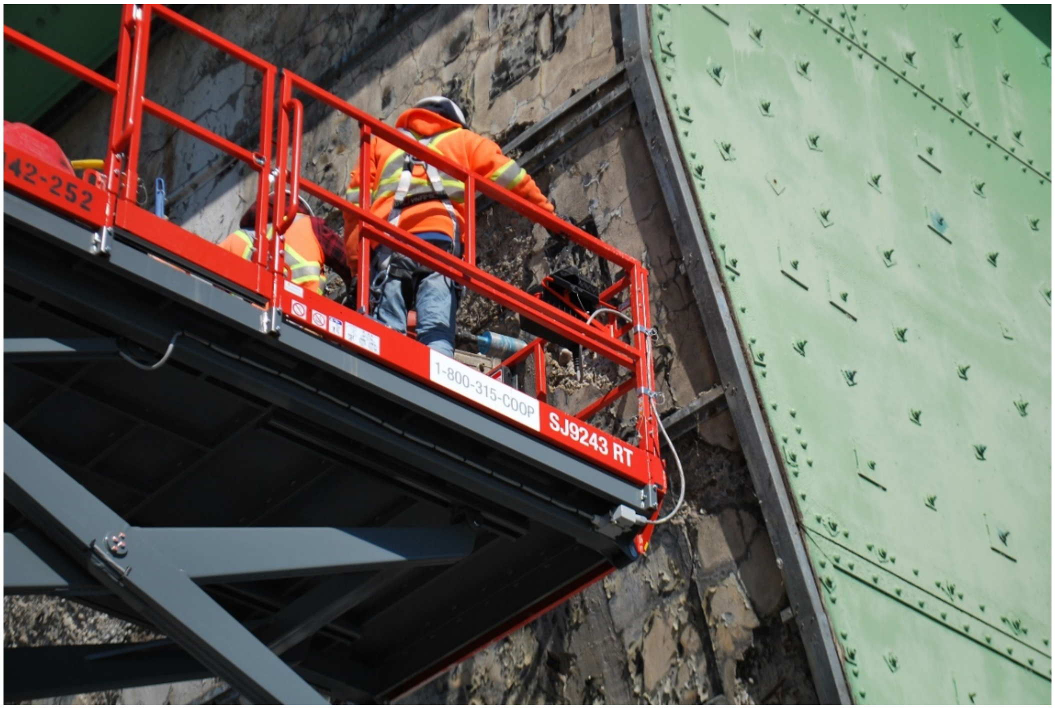


Figure A9 – Concrete coring in progress at core location C2. A core measuring approximately 2600mm was removed from this location. The opposite end of the core reached the South compartment on the East face.

Figure A10 – Concrete core location C2 on West face.

APPENDIX A

Figure A11 - Concrete condition in center portion of West face showing surface delamination, exposed reinforcing steel and steel mesh.

Figure A12 – Top edge of West face. Crack, 25 mm – 50 mm in width running entire top edge of West face. Debris shown is assumed to be from birds nesting.

APPENDIX A

Figure A13 – Spalling at concrete found at the top of the West face (same location as in Figure A.12). Feather shown indicates debris is from birds nesting.

Figure A14 – Steel cladding removed from North and South ends of bottom face. Partial view of West face also shown.

Figure A15 – Condition of concrete on South end of bottom face. Localized areas of cracking, delamination and spalling. Angle iron embedded in concrete is in poor condition showing generalized corrosion.

Figure A16 – Condition of concrete on North end of bottom face. Localized area of cracking, delamination and spalling. Surface staining is observed on remaining steel cladding.

APPENDIX A

Figure A17 – Test pit taken from North East corner of bottom face. Underlying steel reinforcement showing severe signs of corrosion.

Figure A18 – Top face covered in roofing material. Door removed from one of four compartments located on the top face, which houses additional concrete weights.

Figure A19 – Surface scaling on the top face (concrete fragments 10-50mm in diameter).

Figure A20 – Coring location C6 (South East corner of top face).

Figure A21 – Coring at location C7 (North East corner of top face).

Figure A22 –Two compartments located on East face designated as South and North. Steel cladding removed from South side of East face. North compartment is where cores C3 and C4 were removed (see Figure A27).

Figure A23 – Surface spalling on South end of East face

Figure A24 – Bottom left hand corner of East face showing signs of spalling, cracking, exposed reinforcement and areas of corrosion.

Figure A25 – Condition of concrete inside South compartment on East face. Excessive map cracking on the ceiling of compartment resulting in efflorescence. Core hole is that of C2 taken from the West face which protruded the entire thickness of the counterweight.

Figure A26 – Condition of concrete inside North compartment on East face. Excessive map cracking on the ceiling of compartment resulting in efflorescence.

Figure A27 – Core holes C3 (left) and C4 (right) inside North compartment on East face. Please refer to the compartments in Figure A22.

Figure A28 – Core holes C3. Cracking observed along core hole to depths of approximately 385 mm.

Figure A29 – Core hole C1.

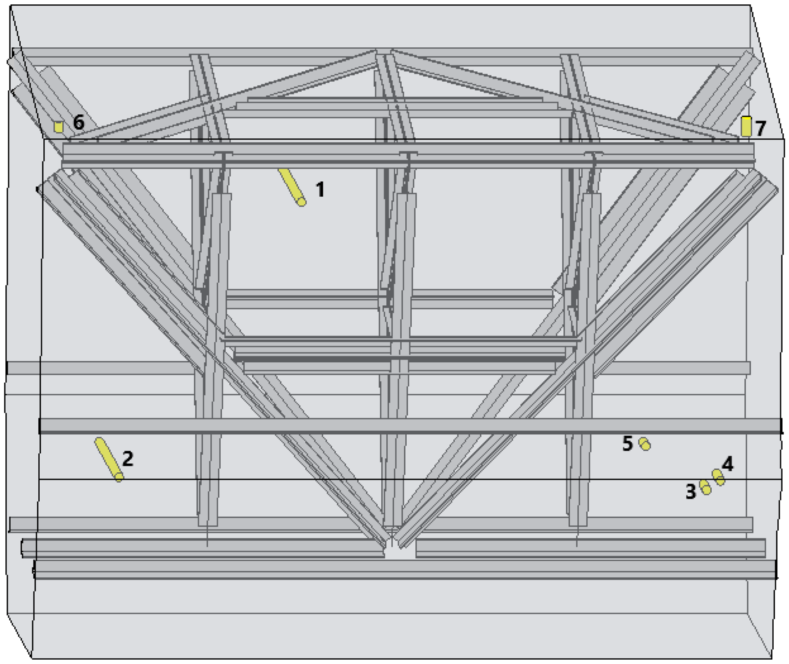
Figure A30 – Core hole C2 (full depth). Photograph taken from inside South compartment on East face. Photograph is looking in the West direction.

APPENDIX

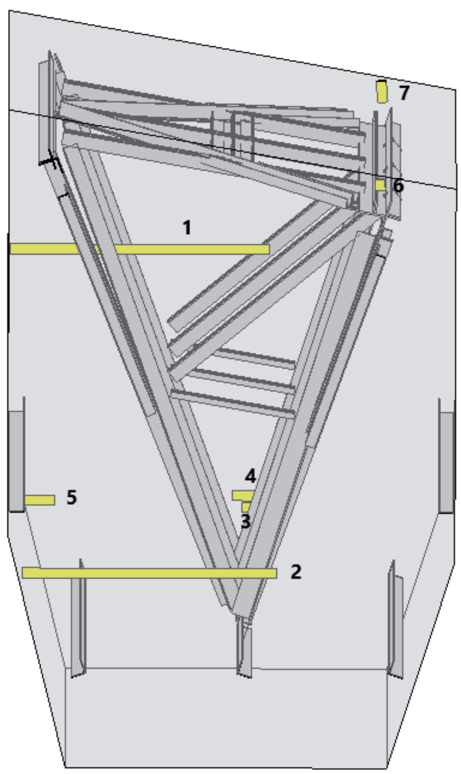
B

CORE LOGS AND LOCATIONS

APPENDIX B



Field core hole locations and extend – isometric view from East



Field core hole locations and extent – isometric view from South

APPENDIX B

Concrete Core Summary

CORE NO.	C1	C2	C3	C4	C5	C6	C7
Face	West	West	East	East	Top	Top	East
Diameter (mm)	100	100	100	100	100	100	100
Length (mm)	2650	2600	480	720	395	220	355
Full Depth (Yes/No)	No	Yes	No	No	No	No	No
Aggregate (Round (R) or Angular (A))	R + A	R + A	R + A	R + A	R + A	R + A	R + A
Aggregate Size (mm)	50-100	50-100	50-100	50-100	50-100	50	50
Cracking (Yes/No)	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Reinforcement/ Steel Present (Yes/No)	Yes	Yes (Steel Plate)	Yes	Yes	No	No	No

Comments:

Core C1:
Approximately 50-75mm concrete removed or lost from the surface prior to coring
Cracking parallel to core length at depths of 160, 300, 535, 570, 695, 1000-1120, 1750, 2030-end
Cold joint at a depth of 1120mm
Sound concrete at depths 300-535, 695-1000 and 1120-2030mm
Reinforcing steel (15M) at depths of 55, 535, 570, 1015 and 2370mm
Visible cracking in aggregates 530mm from the surface

Core C2:
Approximately 100mm concrete removed or lost from the surface prior to coring
Cracking parallel to core length at depths of 250, 300, 490, 519, 540, 840, 1070 (likely from coring), 1815, 2210, 2430, 2540.
Hairline crack perpendicular to core length at 900 and 1330-1570mm
Steel plates (3mm in thickness) at depths of 1000 and 1300mm
Sound concrete at depths 615-840, 1300-2400mm
Reinforcing steel (15M) at depths of 615, 2400, 2525mm
Visible cracking in aggregates 420mm from the surface

Core C3:
Cracking parallel to core length at depths of 20, 220, 235, 330-385mm
Reinforcing steel (15M) at depths 205, 235, 320 (imprint)

Core C4:
Cracking parallel to core length at depths of 40, 280, 350, 560mm
Hairline crack perpendicular to core length at 220-280 and 560mm to end
Reinforcing steel (15M) at depths 695 (imprint)

APPENDIX B

Core C5:
Approximately 80mm concrete removed or lost from the surface prior to coring
Cracking parallel to core length at depths of 80, 100, 185, 200-235, 280mm

Core C6:
Cracking parallel to core length at depths of 50 and 130mm
Underlying plate in good condition showing no signs of generalized corrosion

Core C7:
Cracking parallel to core length at depths of 90-240mm
Underlying plate in good condition showing no signs of generalized corrosion

APPENDIX

C

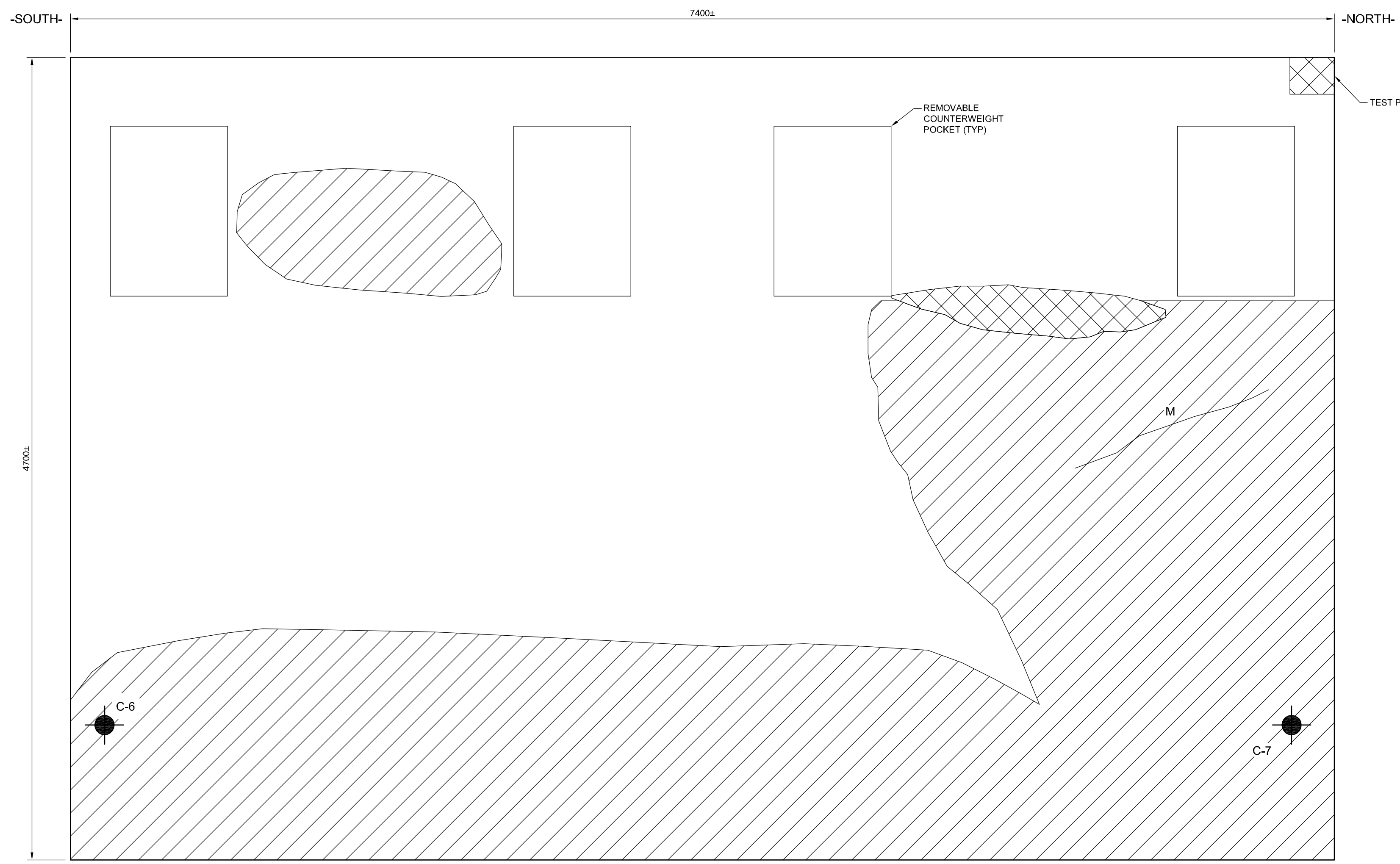
CONDITION DRAWINGS

NOTES:

1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONTRACT DRAWINGS.

LEGEND

- DELAMINATION
- SPALLS
- M — MEDIUM CRACK (0.3 TO 1.0mm)



COUNTERWEIGHT - TOP FACE
1:15

Public Works and Government Services Canada
Architectural and Engineering Services
Ontario Region

Travaux publics et Services gouvernementaux Canada
Services d'architecture et de génie
Région de l'Ontario

wsp

revision		date

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the engineer of all discrepancies.

Ne pas mettre à l'echelle les plans.
Vérifier les dimensions et conditions sur le site et en aviser l'ingenieur immédiatement de tous divergence.

A Detail No. / No. du détail
B drawing no. — where detail required / dessin no. — où détail exigé
C drawing no. — where detailed / dessin no. — où détaillé

project title / titre du projet
LASALLE CAUSEWAY BRIDGE COUNTERWEIGHT REHABILITATION STUDY

drawing title / titre du dessin
CONCRETE COUNTERWEIGHT CONDITION SURVEY TOP FACE

drawn by / dessine par	JL
designed by / conc par	BH
approved by / approuve par	SD
tender / soumission	project manager / administrateur de projets
project date / date du projet	2020/09/21
project no. / no. du projet	R.108246.001
drawing no. / dessine no.	1

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DATE PLOTTED: 12/11/2020 9:52:31 AM BY: REESEB

04 11

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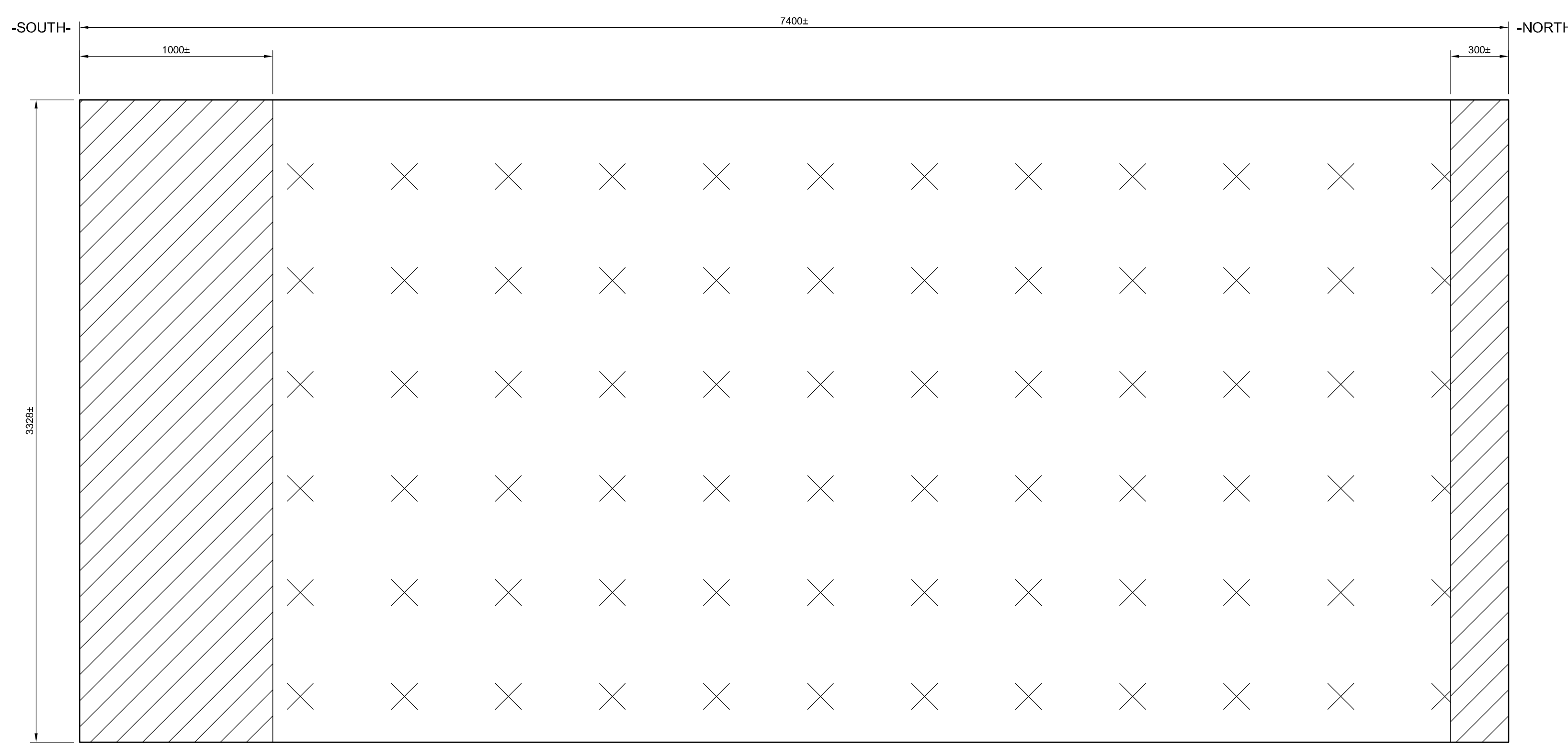
NOTES:

1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONTRACT DRAWINGS.

LEGEND

DELAMINATION

NOT INSPECTED



COUNTERWEIGHT - BOTTOM FACE

1:15

Public Works and Government Services Canada
Architectural and Engineering Services
Ontario Region

Travaux publics et Services gouvernementaux Canada
Services d'architecture et de génie
Région de l'Ontario

wsp

revision		date

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C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
LASALLE CAUSEWAY BRIDGE COUNTERWEIGHT REHABILITATION STUDY

drawing title / titre du dessin
CONCRETE COUNTERWEIGHT CONDITION SURVEY BOTTOM FACE

drawn by / dessine par	JL
designed by / conc par	BH
approved by / approuve par	SD
tender / soumission	project manager / administrateur de projets
project date / date du projet	2020/09/21
project no. / no. du projet	R.108246.001
drawing no. / dessine no.	2

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NOTES:

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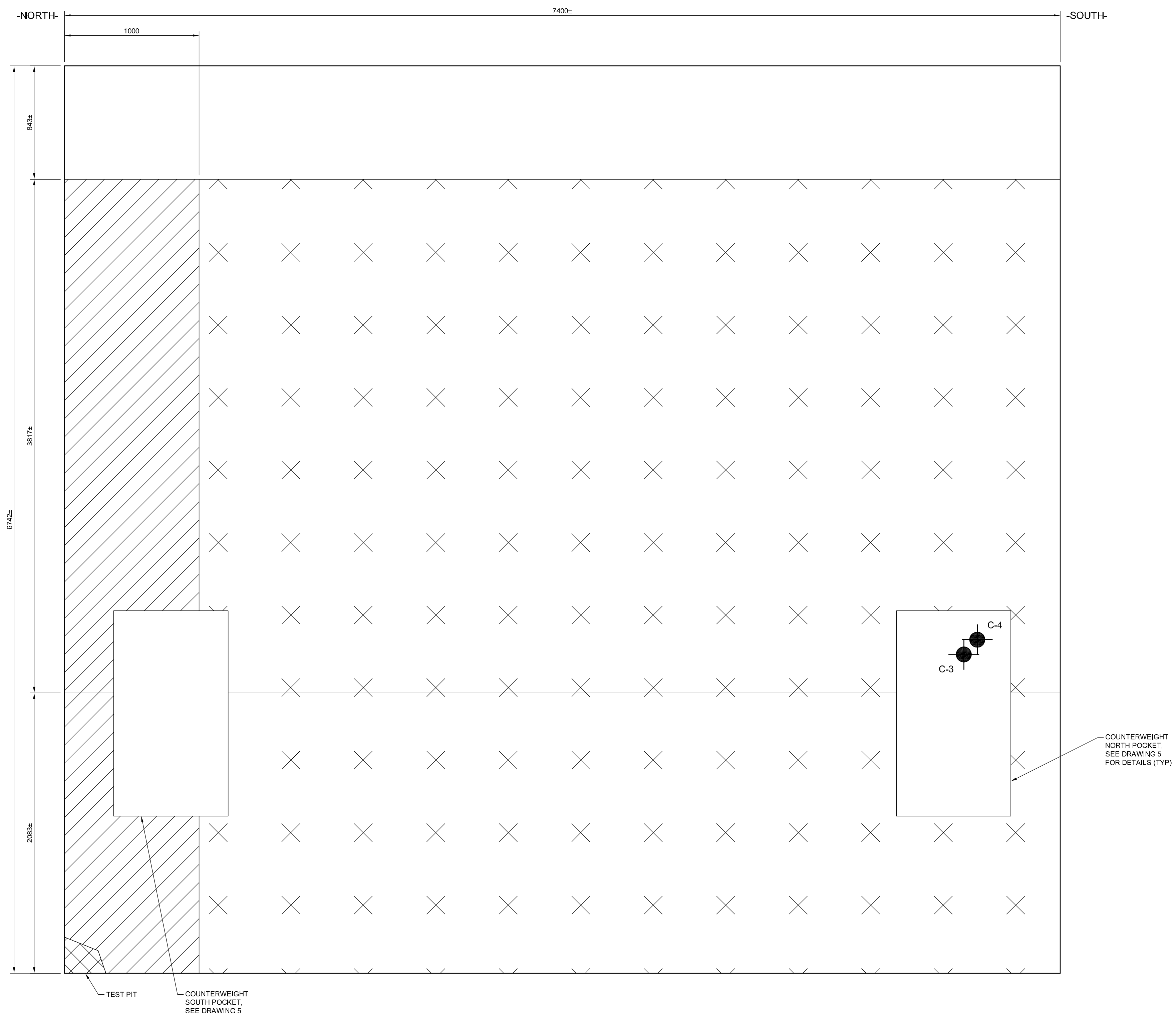
LEGEND

- DELAMINATION
- SPALLS
- NOT INSPECTED



COUNTERWEIGHT - EAST FACE

1:15

Public Works and Government Services Canada
Architectural and Engineering Services
Ontario Region

Travaux publics et Services gouvernementaux Canada
Services d'architecture et de génie
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revision		date

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A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
LASALLE CAUSEWAY BRIDGE
COUNTERWEIGHT REHABILITATION STUDY

drawing title / titre du dessin
CONCRETE COUNTERWEIGHT CONDITION SURVEY EAST FACE

drawn by / dessine par: JL

designed by / conc par: BH

approved by / approuve par: SD

tender / soumission

project manager / administrateur de projets

project date / date du projet: 2020/09/21

project no. / no. du projet: R.108246.001

drawing no. / dessine no.: 3

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NOTES:

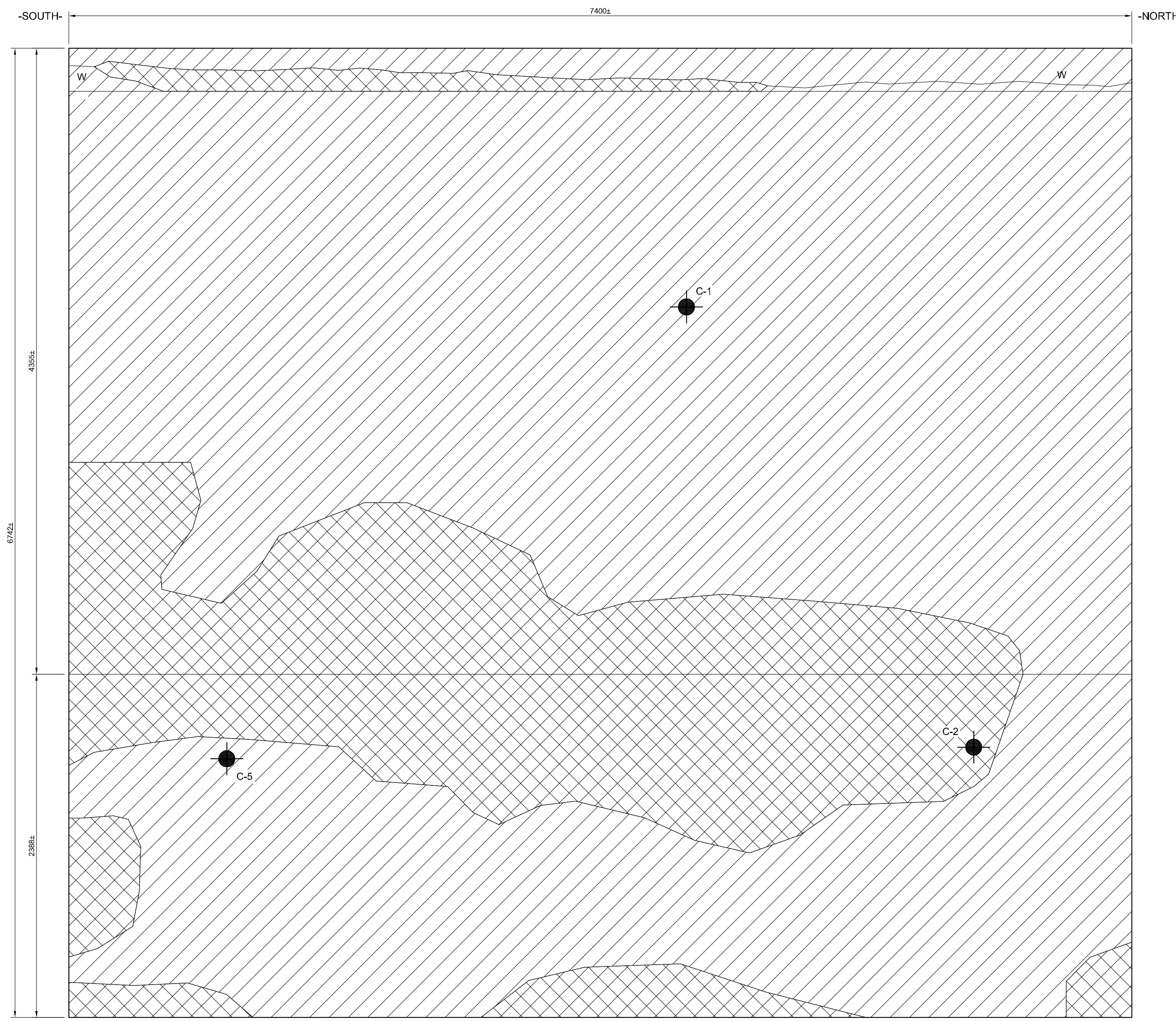
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONTRACT DRAWINGS.

LEGEND

DELAMINATION

SPALLS

W WIDE CRACK (>1.0mm)



COUNTERWEIGHT - WEST FACE
1:15

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Région de l'Ontario

wsp

revision | date

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Ne pas mettre à l'echelle les plans.
Vérifier les dimensions et conditions sur le site et en aviser l'ingenieur immédiatement de tous divergence.

A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
LASALLE CAUSEWAY BRIDGE COUNTERWEIGHT REHABILITATION STUDY

drawing title / titre du dessin
CONCRETE COUNTERWEIGHT CONDITION SURVEY WEST FACE

drawn by / dessine par: JL

designed by / conc par: BH

approved by / approuve par: SD

tender / soumission

project manager / administrateur de projets

project date / date du projet: 2020/09/21

project no. / no. du projet: R.108246.001

drawing no. / dessine no.: 4

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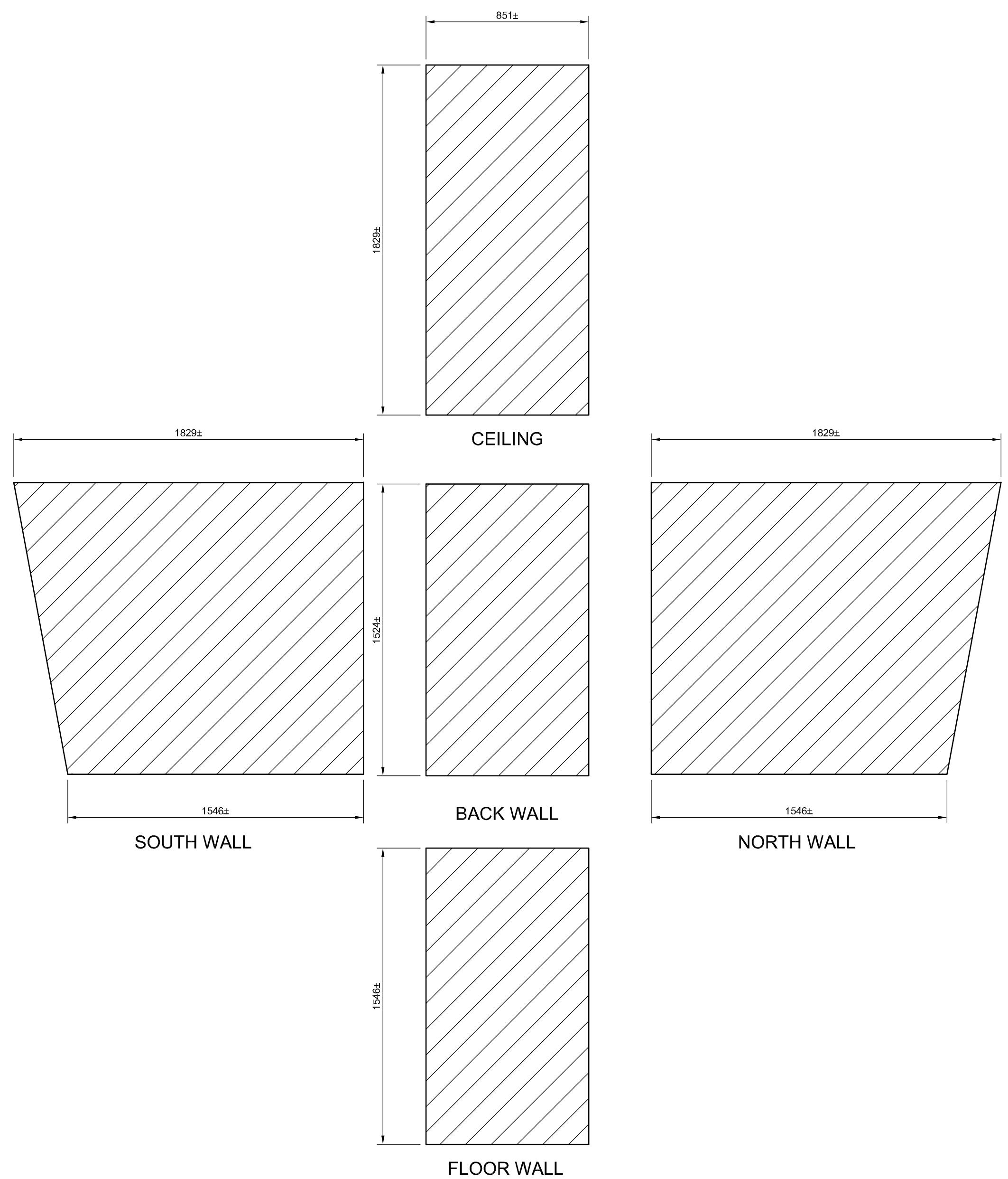
GSC-B1 DATE PLOTTED: PLOT SCALE: CAD FILE:

NOTES:

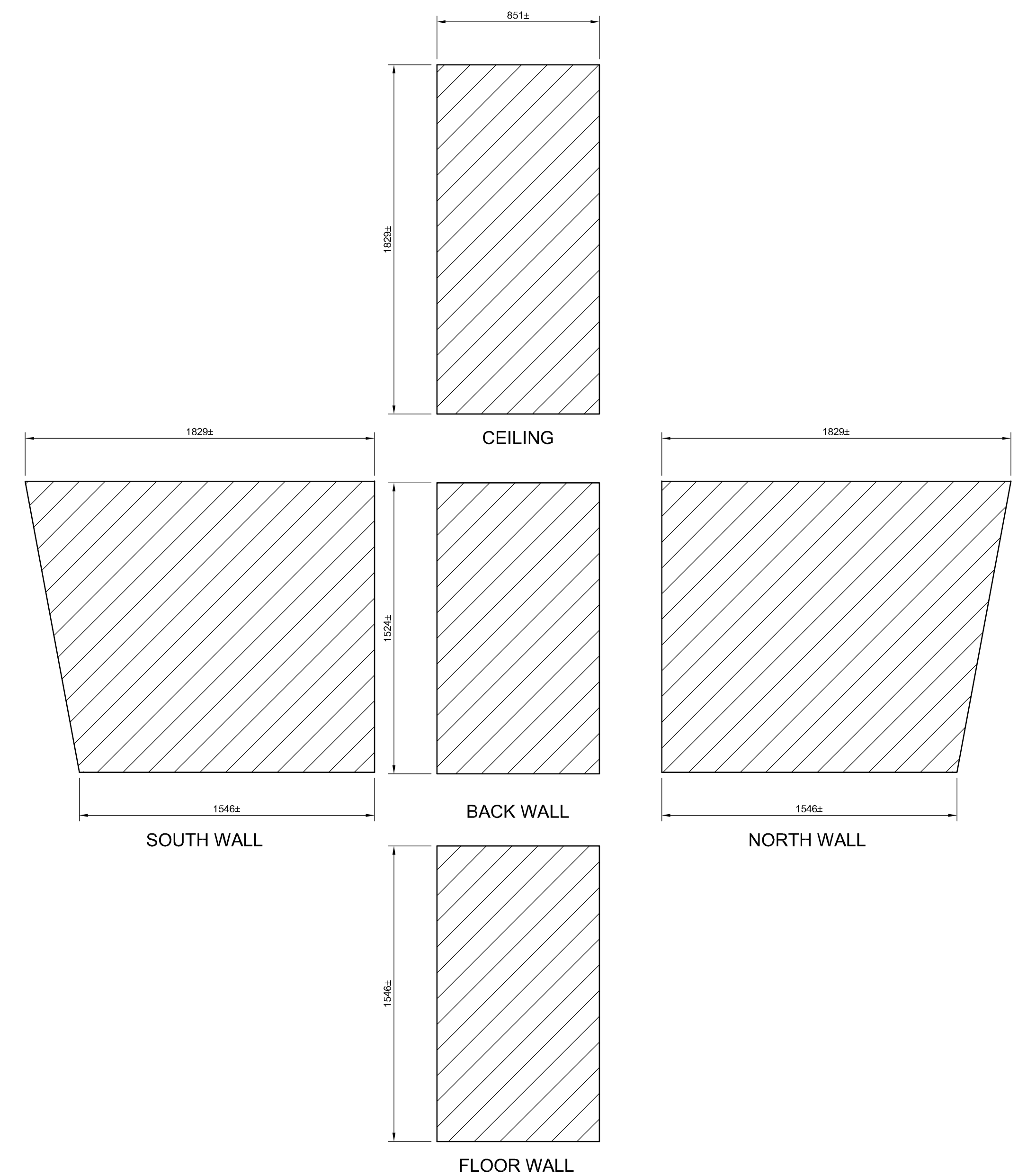
1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONTRACT DRAWINGS.
2. IN ADDITION TO DELAMINATION, MAP CRACKING AND EFFLORESCENCE IS PRESENT ON THE CEILING OF BOTH COMPARTMENTS. MEDIUM SCALING IS FOUND ON ALL POCKET WALLS.

LEGEND

DELAMINATION



COUNTERWEIGHT - NORTH POCKET
1:15



COUNTERWEIGHT - SOUTH POCKET
1:15

Public Works and Government Services Canada
Architectural and Engineering Services
Ontario Region

Travaux publics et Services gouvernementaux Canada
Services d'architecture et de génie
Région de l'Ontario

wsp

revision		date

Do not scale drawings.
Verify all dimensions and conditions on site and immediately notify the engineer of all discrepancies.

Ne pas mettre à l'echelle les plans.
Vérifier les dimensions et conditions sur le site et en aviser l'ingenieur immédiatement de tous divergence.

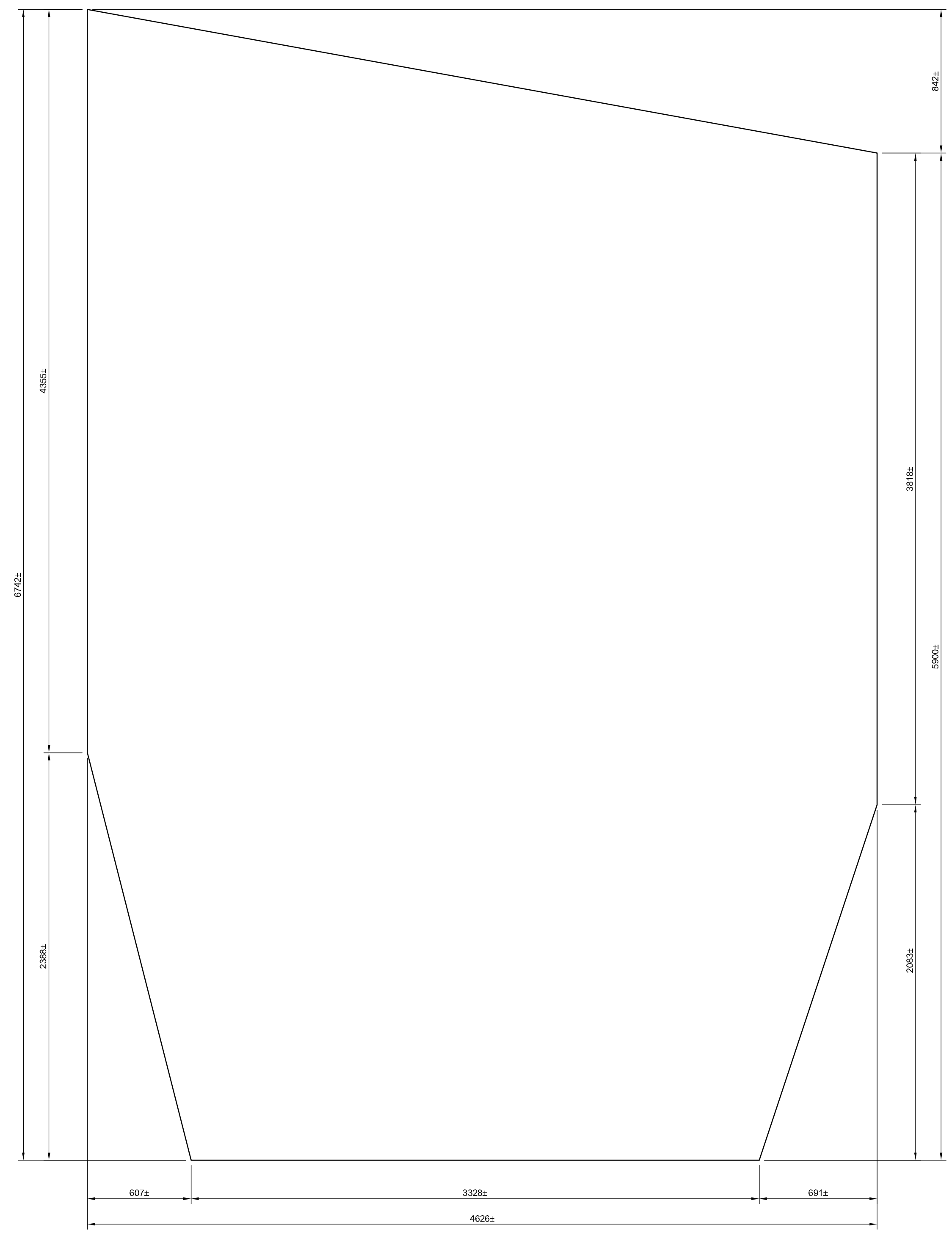
A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet	LASALLE CAUSEWAY BRIDGE COUNTERWEIGHT REHABILITATION STUDY
drawing title / titre du dessin	CONCRETE COUNTERWEIGHT CONDITION SURVEY COUNTERWEIGHT POCKETS
drawn by / dessine par	JL
designed by / conc par	BH
approved by / approuve par	SD
tender / soumission	project manager / administrateur de projets
project date / date du projet	2020/09/21
project no. / no. du projet	R.108246.001
drawing no. / dessine no.	5

CAD FILE LOCATION AND NAME: M:\20M-01325-00 LaSalle Causeway Bridge\SHEETS\001 COUNTERWEIGHT INSPECTION\20M-01325-00-005.dwg
MODIFIED: 12/11/2020 9:58:15 AM BY: REESEB
DATE PLOTTED: 12/11/2020 9:58:40 AM BY: REESEB
GSC-B1 DATE PLOTTED: PLOT SCALE: CAD FILE:

NOTES:

1. THIS DRAWING SHALL BE READ IN CONJUNCTION WITH ALL OTHER CONTRACT DRAWINGS.
2. INSPECTION OF NORTH AND SOUTH FACE NOT CONDUCTED DUE TO STEEL PLATE COVERING ENTIRE FACE.



COUNTERWEIGHT - NORTH FACE
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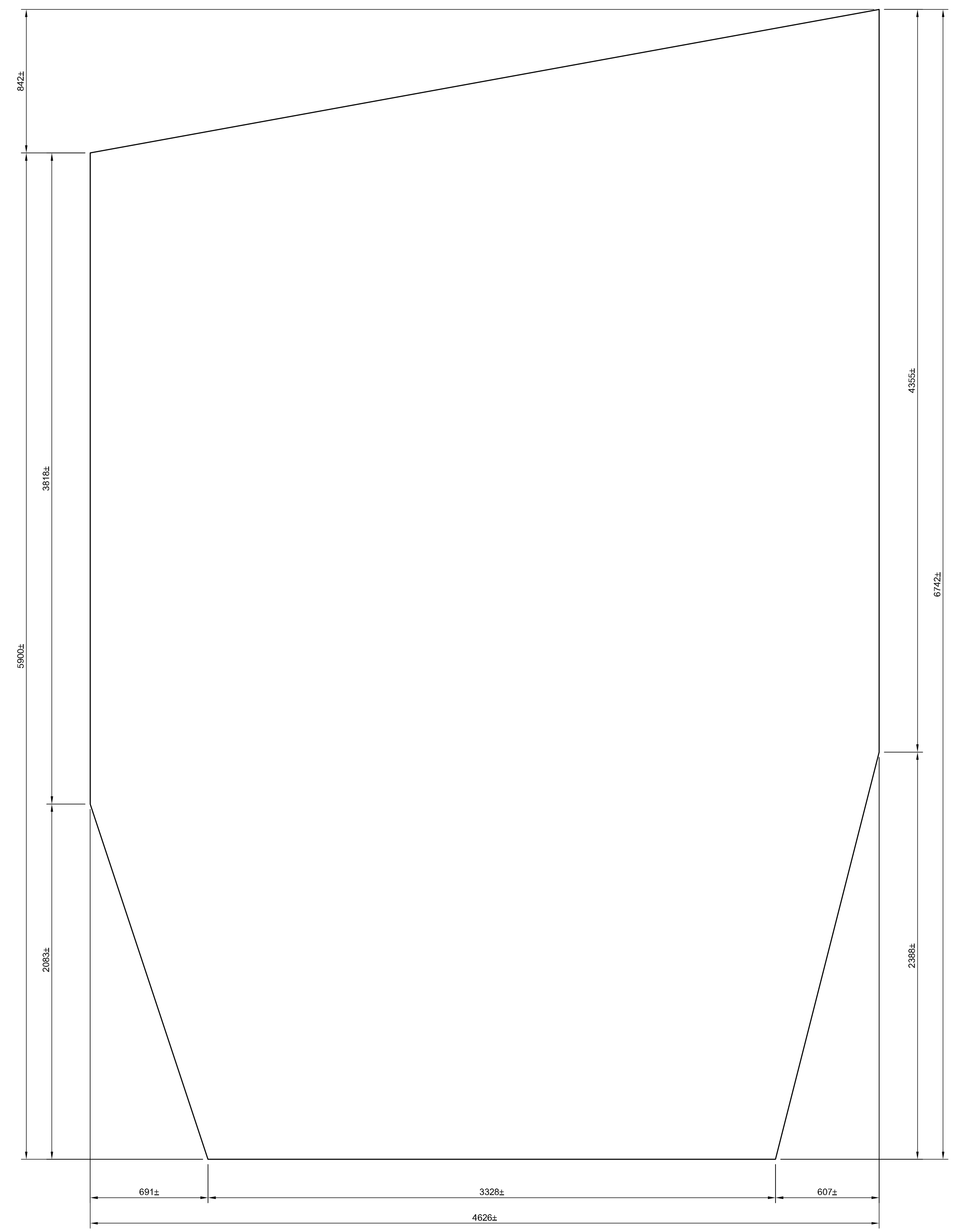

COUNTERWEIGHT - SOUTH FACE
1:15

Public Works and Government Services Canada
Architectural and Engineering Services
Ontario Region

Travaux publics et Services gouvernementaux Canada
Services d'architecture et de génie
Région de l'Ontario

revision		date

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Verify all dimensions and conditions on site and immediately notify the engineer of all discrepancies.

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A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
LASALLE CAUSEWAY BRIDGE
COUNTERWEIGHT REHABILITATION STUDY

drawing title / titre du dessin
CONCRETE COUNTERWEIGHT
CONDITION SURVEY
NORTH AND SOUTH FACE

drawn by / dessine par: JL

designed by / conc par: BH

approved by / approuve par: SD

tender / soumission

project manager / administrateur de projets

project date / date du projet: 2020/09/21

project no. / no. du projet: R.108246.001

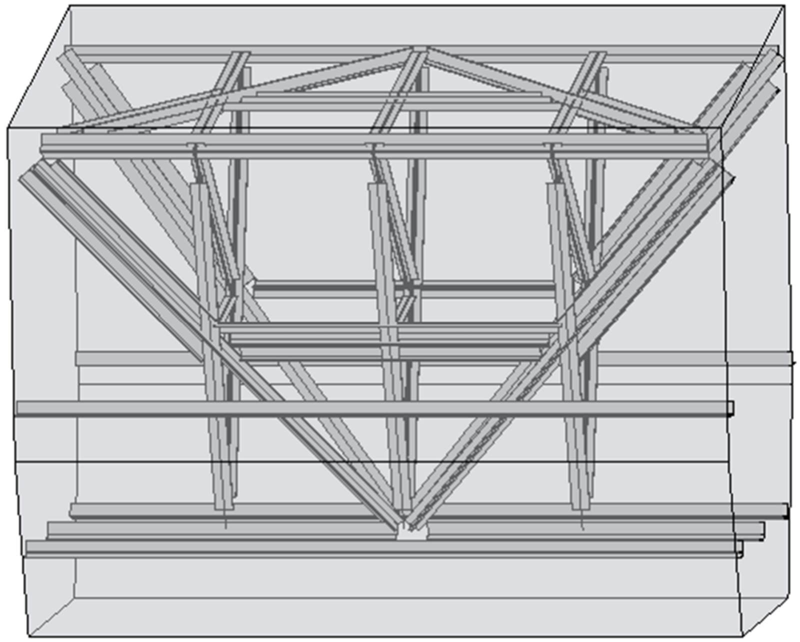
drawing no. / dessine no.: 6

APPENDIX

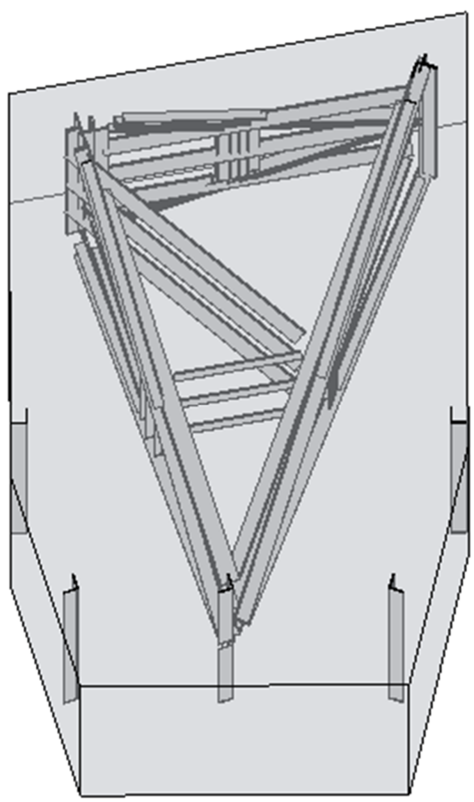
D

3D MODEL / DRAWINGS

APPENDIX D

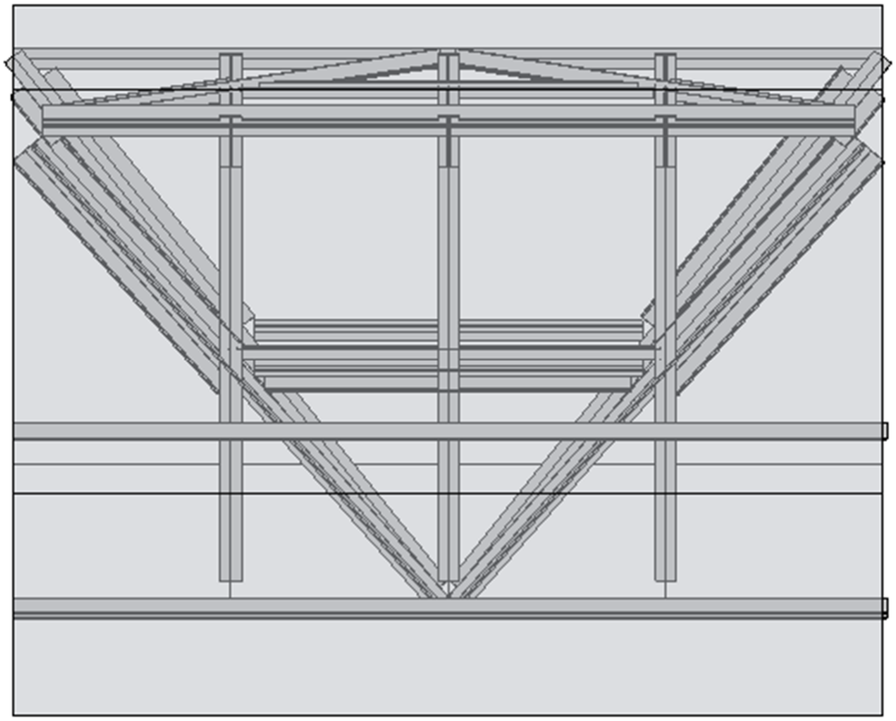


View 1 – Isometric view from East side

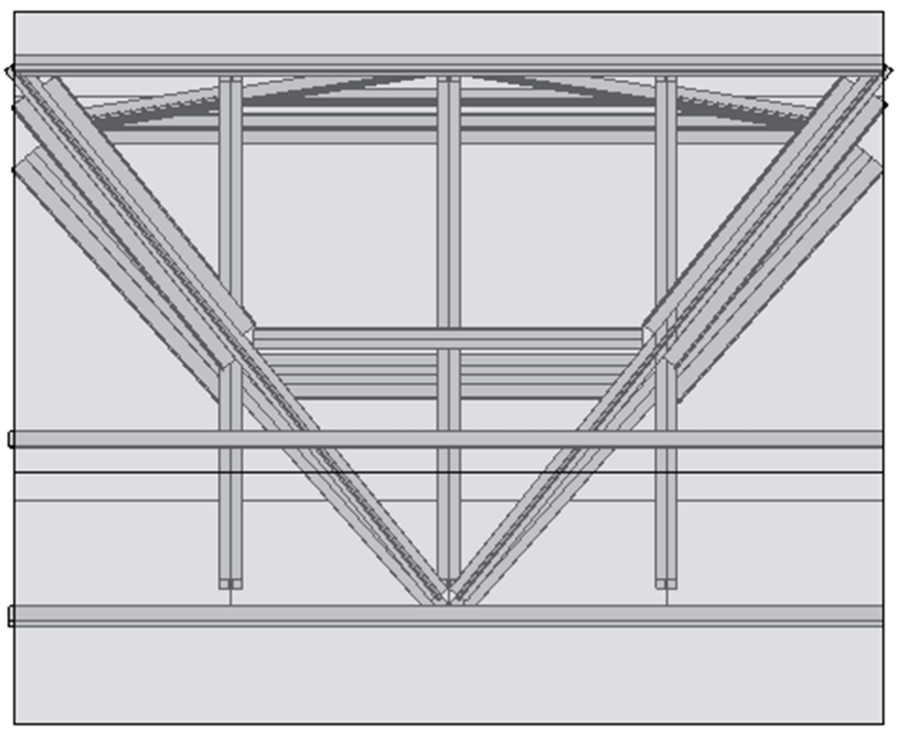


View 2 – Isometric view from North side

APPENDIX D

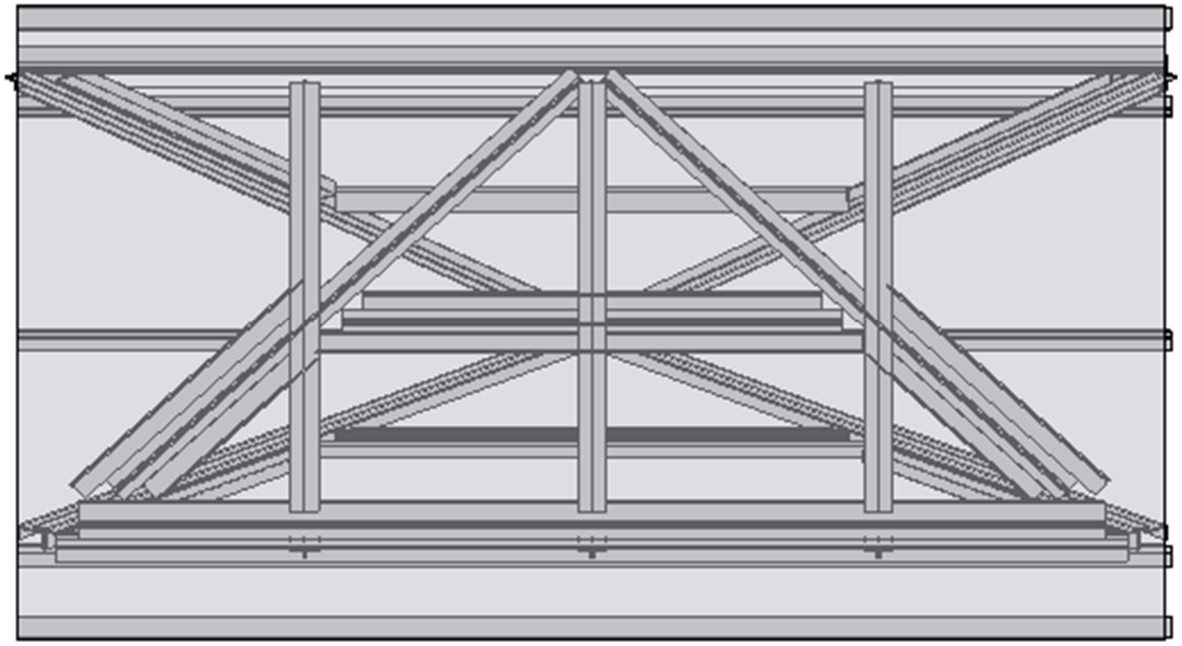


View 3 – East Face

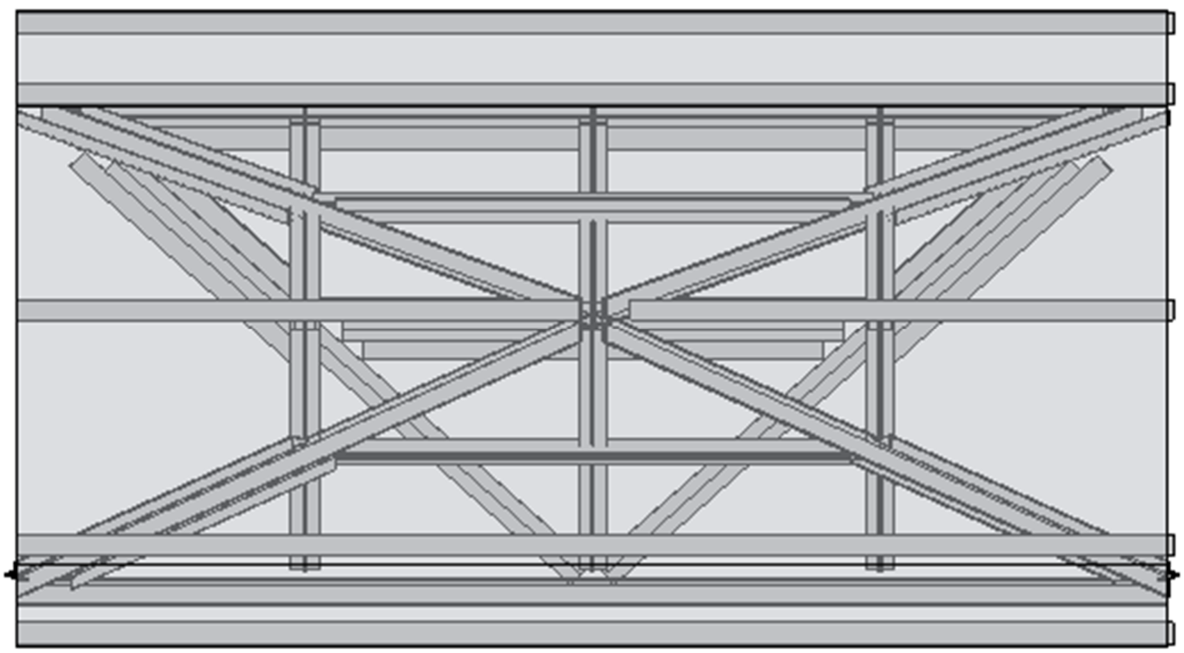


View 4 – West face

APPENDIX D

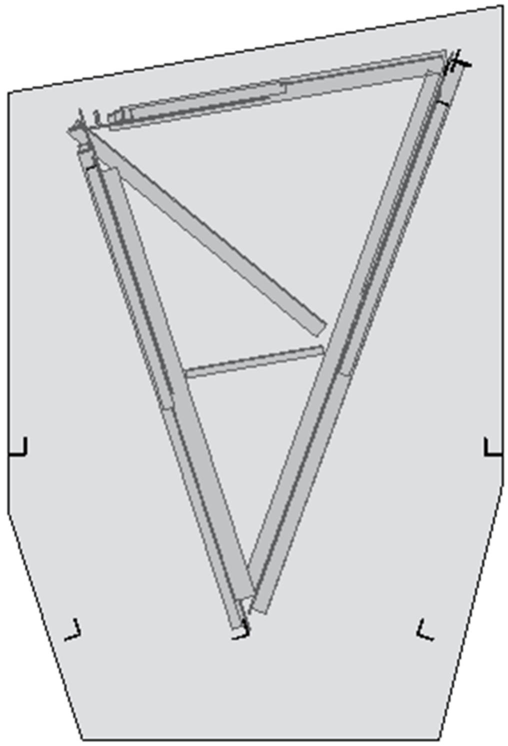


View 5 – Top face

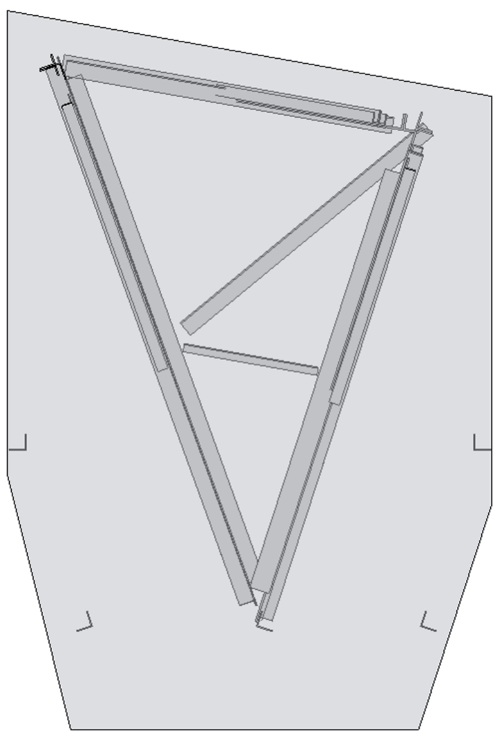


View 6 – Bottom face

APPENDIX D



View 7 – North face



View 8 – South face

APPENDIX

E

PETROGRAPHY REPORT

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December 10, 2020

Study of concrete from counterweight of Bascule Bridge at La Salle Causeway, Kingston, Ontario

Background

This structure was completed in 1917. The concrete in the counterweight is undergoing a condition survey. As part of this study a petrographic examination of the concrete was undertaken.

Samples

A summary description of the cores delivered for study and dimensions is shown in Table 1.

Table 1: Summary of core numbers and dimensions

Core identification	Diameter and length of polished face	Notes
C1	95 x 230 mm	Core taken from close to middle of a longer core through the counterweight.
C2	95 x 170 mm	Core taken from close to end (near the exterior surface) of a longer core through the counterweight. Core showed considerable cracking and required gluing with epoxy to enable sample preparation

Sample Preparation

The cores were cut length ways with a diamond saw. They were polished with a variety of abrasives to produce a flat polished face. Photographs of the polished cores are shown in figures 1 and 2.

Techniques

The polished surface of each core was examined under a binocular microscope in reflected light and a Damage Rating Index (DRI) established for each core (see Annex to this report). Examination was done at a magnification of 16x but for detailed examination of difficult to identify features, magnification up to about to 50x was used. The results are shown in tables 2 and 3 with a summary in table 4.

Following examination of the polished surface, the other half of each core was broken with a hammer and the broken fragments examined under a binocular microscope in reflected light to identify secondary minerals and other features. Where necessary, materials were extracted from the cores, crushed and mounted on glass slides as powders. The powders were immersed in refractive index oils of various values and covered with a thin cover glass. These were then examined using a petrographic microscope at from about 30x to 400x magnification using both plane polarized and cross polarized light. This allowed the optical properties of the minerals and substances to be determined in a effort to determine the nature and composition of the material. The procedures outlined in ASTM C 856 "Standard practice for petrographic examination of hardened concrete" were used as a guide in conducting the examination.

Overall concrete condition - all cores

The concrete in both cores was generally poorly proportioned and moderately well consolidated. Core 1 showed a deficiency in coarse aggregate and some areas deficient in cement paste resulting in voids around poorly cemented fine aggregate. Core 2 showed an excess of coarse aggregate. On cutting with the diamond saw a clean cut was obtained with core 1 but core 2 showed fragmentation due to existing cracking.

When broken with a hammer, there were many coarse aggregate/mortar sockets and few broken coarse aggregate particles indicating a generally poor coarse aggregate/mortar bond. The bond of paste to the fine aggregate was judged to be poor with few broken sand grains. The concrete in core 1 was judged to be of moderate strength based on the response when hit with a hammer and weak in core 2.

<u>Coarse aggregate</u>

The coarse aggregate (material > 5mm) made up about 15 to 20 per cent by volume of the concrete in core 1 and 45 to 50 per cent in core 2 and was poorly graded. The maximum particle dimension in both cores was about 40 to 50 mm.

Core 1 - The particles were a mixture of uncrushed rounded carbonate and lesser amounts of rounded red and pink coloured granite gravel. The carbonate was a mixture of fine grained light grey and medium grey coloured limestone. Some of the limestone was siliceous and dolomitic sometimes with small dolomite rhombohedra characteristic of the texture of alkali-carbonate reactive varieties. The limestone contained fossils and pelloidal types indicating it was probably from rocks of Middle Ordovician age that outcrop near Kingston and also in the Ottawa / St Lawrence Lowlands of eastern Ontario.

Core 2 - The particles were a mixture of rounded gravel and crushed limestone and dolomitic limestone making up about 90 per cent of the coarse aggregate. There were also some particles of rounded limestone gravel smaller than about 20 mm. The dolomitic limestone was a light to medium grey coloured fine grained material sometimes with rounded quartz sand grains of about 0.5 to 1 mm diameter dispersed through the carbonate matrix. Granite and other siliceous rock made up about 10 per cent. There were darkened reaction rims on the outside edge of the fractured dolomitic limestone particles indicating a possible reaction with the alkalies in the cement. No signs of formation of calcite, that is often indicative of alkali-carbonate rock reaction, adjacent to the coarse aggregate dolomitic limestone particles were observed.

The majority of coarse aggregate particles were sound, unweathered and judged suitable for use as a concrete aggregate. It should be noted that some coarse aggregate particles in core 2 were severely cracked. The coarse aggregate in both cores was probably from the same source but was segregated that gave variations in composition and proportion.

Fine aggregate

The fine aggregate (material < 5mm) in core 1 made up about 20 to 25 per cent by volume. In core 2 fine aggregate made up about 45 to 50 per cent by volume. The particles had a rounded to angular shape. The fine aggregate was judged to be well graded, consisting of sound and strong particles.

The fine aggregate was made up of about 70 % quartz and feldspar and about 20 % carbonate and siliceous rock fragments (biotite gneiss, granite and diabase). The quartz was usually smaller than 1 mm in diameter and was sometimes spherical in shape. Spherical quartz grains are somewhat unusual but are derived in eastern Ontario from bedrock of the Nepean formation of probably Cambrian or early Ordovician age that outcrops to the east of Kingston and in the Ottawa - St Lawrence Lowlands. There were trace amounts (< 1 % each) of calcite crystals, friable white sandstone (often with dark rims), sandstone with a dark coloured cement, magnetite, purple or violet coloured garnet, wood fragments, cotton waste, coal and biotite mica.

Cement paste

The cement paste made up about 20 to 25 per cent of the volume of the concrete and had an even light grey colour. In thin edges the paste had a dull appearance. The extent of carbonation was tested with phenolphalein indicator. The concrete was uncarbonated as far as could be determined in core 2 and was uncarbonated throughout core 1.

Based on scratch hardness and appearance the water/cement ratio was judged to be more than 0.5.

Voids

The concrete in both cores was non air-entrained. The amount of voids was hard to determine because of cement being absent from some areas of the concrete leaving irregular shaped voids. In both cores, the voids were estimated to be less than about 5 per cent.

The majority of the voids in both cores had a thin white lining of needle like ettringite crystals growing perpendicular to the air void wall. The needles were about 25 to 50 µm long. Ettringite

($3CaO.Al_2O_3.3CaSO_4.31H_2O$) was proved by having a refractive index just < 1.47 and parallel extinction in X polars. The amount was not excessive. Small amounts of very small Portlandite ($Ca[OH]_2$) and calcium carbonate ($CaCO_3$) particles were also found within air voids. These three minerals are a normal product of the hydration of Portland cement and not of concern when not present in excessive amounts.

<u>Cracks</u>

Generally the concrete in core 1 was remarkably uncracked. A few minor cracks in the mortar and in the coarse aggregate were noted in the Damage Rating Index (Table 2) part of the examination. These cracks were no wider than 0.05 mm.

Core 2 had many cracks as shown in Figure 2. The cracks were partly lined with calcium carbonate and ettringite. The crack width was estimated to range from 0.1 mm to 1.0 mm. The cracks went both around aggregate particles and through them. The major cracking in this core was roughly parallel to the outer surface and because of that were most probably caused by freezing and thawing.

<u>Embedded items</u>

No embedded items were found within the concrete.

Discussion

The Damage Rating Index (DRI) results are shown in Tables 2 and 3 and show values of 49 and 440. A value of about 50 indicates that the concrete is microscopically in generally reasonably good condition with mild signs of damage due to alkali-aggregate reaction. A DRI value of 440 indicates considerable cracking and damage to the concrete leading to significant reduction in modulus and loss of strength (See Annex). The cause of the damage is certainly due in part to alkali-aggregate reaction but freezing and thawing of this non air-entrained concrete in core 2 also played a significant role especially earlier in the life of the concrete. This probably occurred before it was given the present metal cladding that would have resulted in reducing water contact with the concrete. Prior to the cladding being applied, the concrete was probably periodically critically saturated and as a result damaged by freezing and thawing every winter. The cracks in core 2 (figure 2), that are roughly parallel to the outer surface, are strongly suggestive of freeze-thaw damage. It seems probable that, provided the majority of water is excluded, future damage due to freezing and thawing will be minimized or eliminated.

Conclusions

1. The concrete in both cores was generally poorly proportioned and moderately well consolidated. Core 1 showed a deficiency in coarse aggregate and some areas deficient in cement paste resulting in voids around poorly cemented fine aggregate. Core 2 showed an excess of coarse aggregate.

2. The majority of the coarse and fine aggregate particles were sound, unweathered and judged suitable for use as a concrete aggregate.
3. The concrete in both cores was non air-entrained.
4. There were minor signs of alkali-aggregate reaction of the coarse aggregate.
5. The Damage Rating Index (DRI) results are shown in Tables 2 and 3 and show values of 49 and 440. Values of about 50 indicate that the concrete is microscopically in generally reasonably good condition with mild signs of damage due to alkali-aggregate reaction. A DRI value of 440 indicates considerable cracking and damage to the concrete.
6. The cause of the damage found in core 2 is certainly due in part to alkali-aggregate reaction but freezing and thawing of this non air-entrained concrete also played a significant role.

Chris Rogers.

Chris Rogers

Table 2: Results of microscopic examination for Damage Rating Index.

Sample Information: Core 1, December 5, 2020.

FEATURE	TOTAL COUNT	FACTOR	Contribution to DRI
Cracks in coarse aggregate	39	0.25	9.75
Open crack in coarse aggregate	2	2	4
Crack with reaction product in coarse aggregate	36	2	72
Disaggregated/corroded aggregate particle	0	2	0
Coarse aggregate debonded	0	3	0
Crack in mortar	6	3	6
Crack with reaction product in mortar matrix	0	3	0
Air void containing AS gel	0	No value in DRI	-
Reaction rim	4 (on white sandstone)	No value in DRI	-
Porous areas deficient in cement	22	No value in DRI	-
Total (Uncorrected Damage Rating Index)			91.75
Total area of sample viewed 9 x 21 = 189 cm^2			189 cm^2
DRI, normalized to 100 cm^2			48.5

Note: "Reaction product" may be alkali-silica gel or other secondary material of unknown composition.

.

Table 3: Results of microscopic examination for Damage Rating Index.

Sample Information: Core 2, December 6, 2020.

FEATURE	TOTAL COUNT	FACTOR	Contribution to DRI
Cracks in coarse aggregate	45	0.25	11.25
Open crack in coarse aggregate	38	2	76
Crack with reaction product in coarse aggregate	82	2	164
Disaggregated/corroded aggregate particle	0	2	0
Coarse aggregate debonded	4	3	12
Crack in mortar	50	3	150
Crack with reaction product in mortar matrix	22	3	66
Air void containing AS gel	1	No value in DRI	-
Reaction rim	11	No value in DRI	-
Porous areas deficient in cement	0	No value in DRI	-
Total (Uncorrected Damage Rating Index)			479.25
Total area of sample viewed 3 + 11 + 11 +15 + 15 + 15 + 14 + 13 + 12 = 109 cm^2			109 cm^2
DRI, normalized to 100 cm^2			439.7

Note: "Reaction product" may be alkali-silica gel or other secondary material of unknown composition.

Table 5: Summary of Damage Rating Index (DRI).

Sample number	CA with crack	CA with open crack	CA with crack + RP	Disagg coarse agg.	CA debonded	Crack in mortar	Crack in mortar + RP	Area exam cm^2	DRI
Weighted value factors used in calculation	*0.25*	*2*	*2*	*2*	*3*	*3*	*3*	-	-
Core 1	9.75	4	72	0	0	6	0	189	48.5
Core 2	11.25	76	164	0	12	150	66	109	439.7

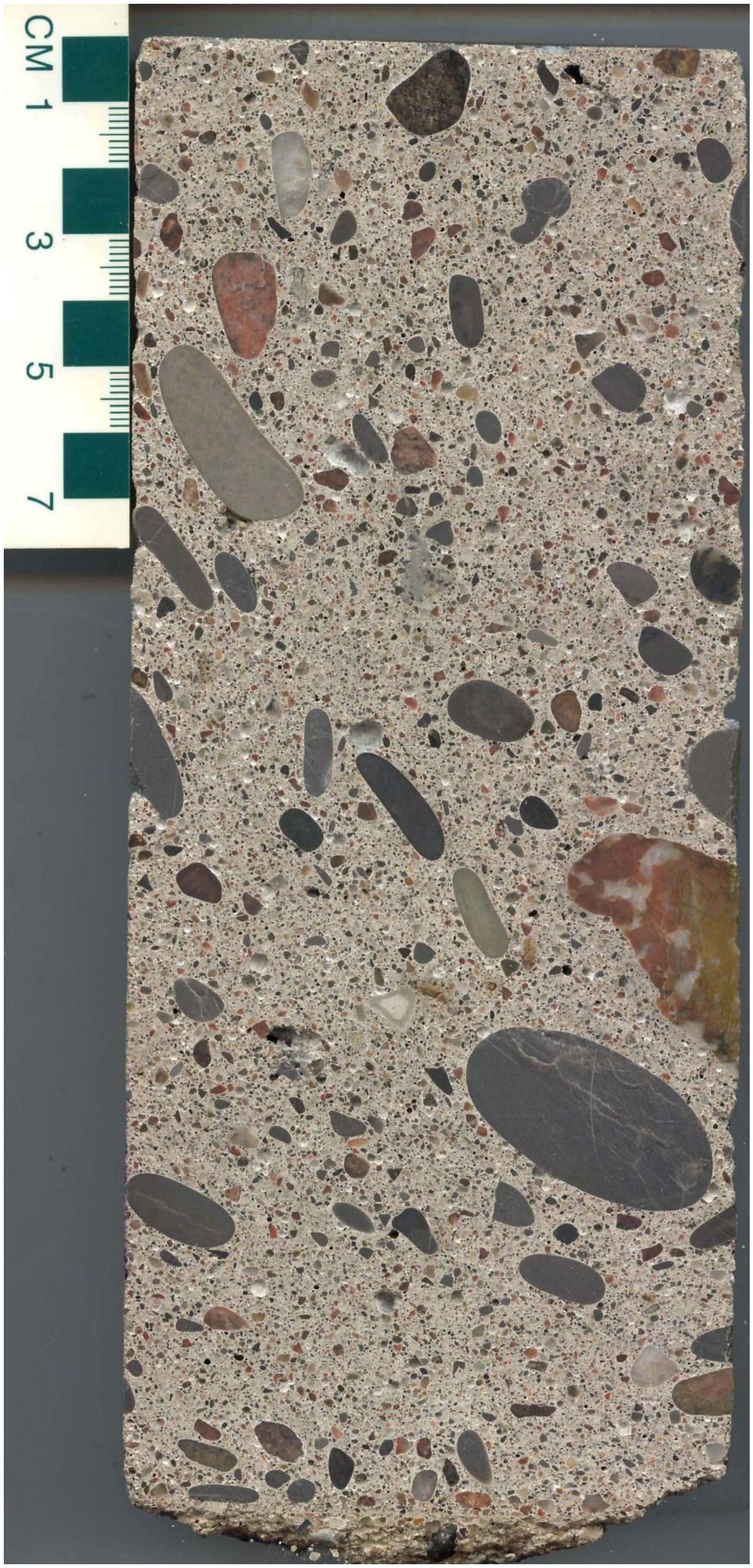

Figure 1: Core 1. Note preferred orientation of long axis of coarse aggregate particles.

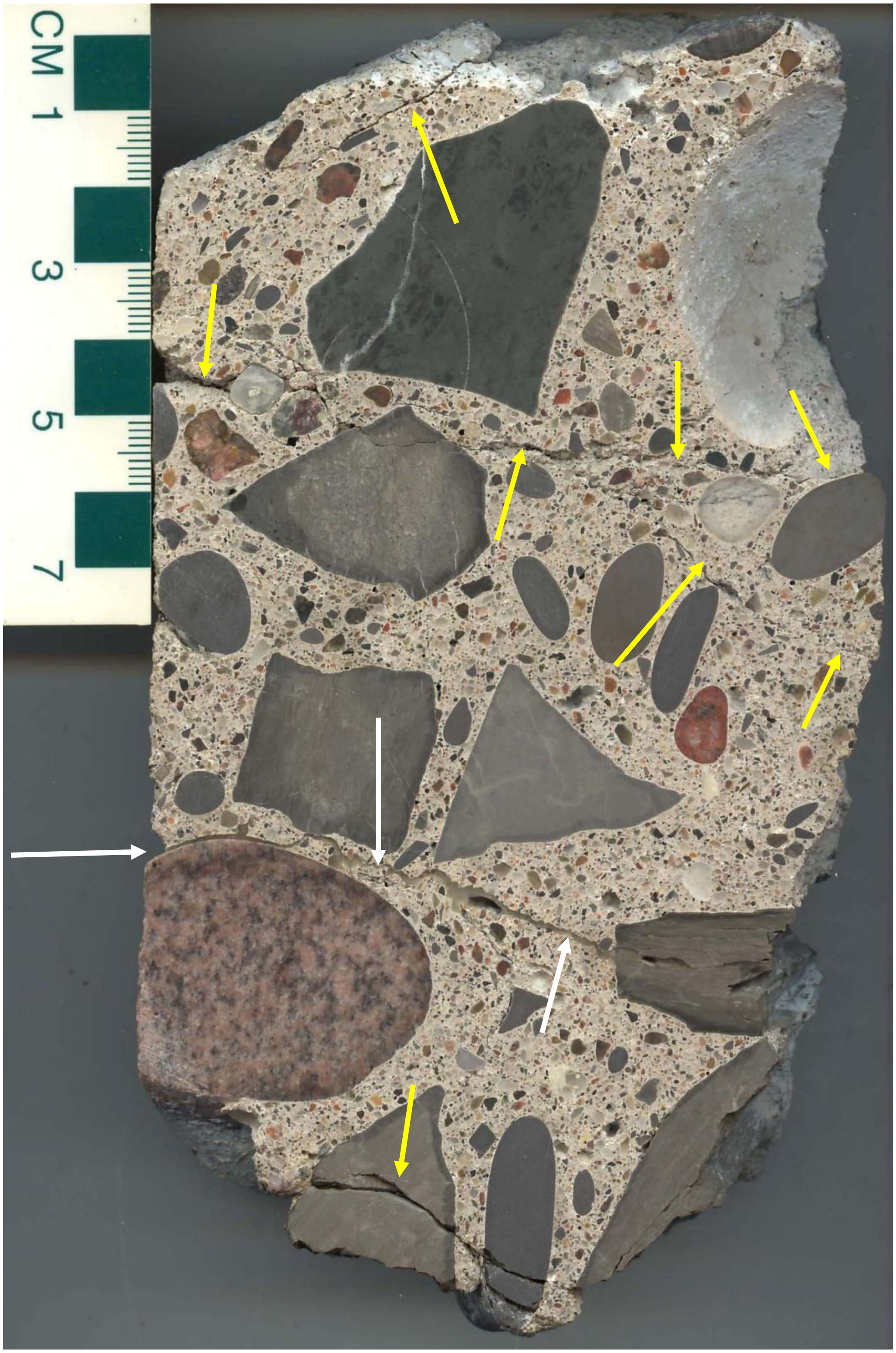

Figure 2: Core 5. White arrows show location of crack that was glued with epoxy during sample preparation. Yellow arrows indicate other significant cracks.

Annex

Damage Rating Index (DRI) on concrete

Background

This petrographic procedure was developed by P. Grattan-Bellew of the National Research Council in Ottawa (Grattan-Bellew and Danay, 1992, Grattan-Bellew, 2012 and Sanchez et al, 2015).

Techniques

Concrete samples (often cores) are cut with a diamond saw into slabs about 40 – 50 mm thick. The cut surfaces are polished with a variety of abrasives on a rotary lap.

Each polished sample face is divided into areas of 10 mm x 10 mm and for each 10 mm square the concrete is examined under a stereomicroscope at about 16 x magnification and the presence of various kinds of defects recorded. The DRI procedure used was that described by Sanchez *et al* 2015.

Discussion

A high DRI does not mean the concrete is suffering from alkali-aggregate related damage but shows a high number of defects that could be caused by a variety of mechanisms such as alkali-aggregate reactions as well as freezing and thawing. However a structure damaged by AAR will have a DRI ranging from about 50 for a mildly affected structure to up to about 1000 for a structure that has been badly affected. Values less than about 50 for concrete more than 40 years old indicate that the concrete is microscopically in generally reasonably good condition (strength excepted).

Grattan-Bellew (2012) in a review of the application of DRI concluded the following:

> "*For a number of reasons determining the critical DRI that is indicative of significant deterioration of the concrete poses a difficult problem. Tentatively, DRI's of greater than ~50 are considered to indicate significant deterioration of the concrete in the structure. However, at present due to the large differences in DRI's determined by different operators it is probably not possible to determine a critical value that would apply to DRI's of all operators."*

Sanchez et al (2017) shows a figure 4, reproduced below, that shows the relationship between DRI and expansion of concrete. This data is derived from laboratory studies where there was no exposure to freezing and thawing. It shows that as expansion increases there is an increase in DRI.

Sanchez et al (2016) shows a figure 3, reproduced below, that shows the relationship between expansion and some properties of hardened concrete. This data is derived from laboratory studies where there was no exposure to freezing and thawing. It shows that as expansion increases, modulus is reduced significantly. Reduction in compressive strength is reduced but not to the same extent.

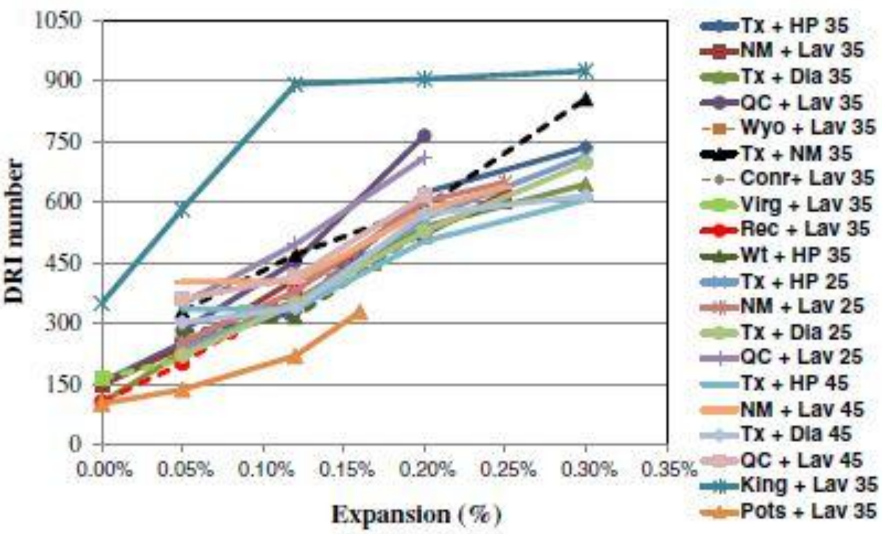

Fig. 4. DRI number for all mixtures analyzed in this study.

Figure 4, from Sanchez et al, 2017.

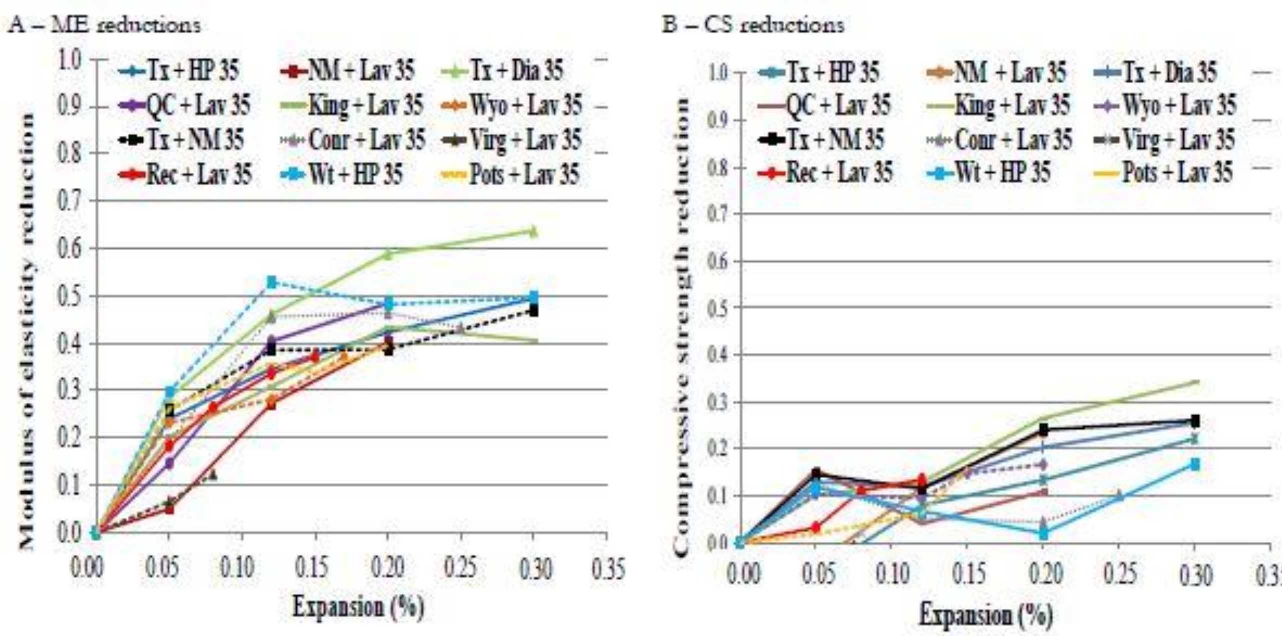

FIGURE 3: Reductions in the modulus of elasticity (ME) (A) and compressive strength (CS) (B) as a function of ASR expansion, for 35 MPa concretes incorporating a variety of reactive aggregates.

Figure 3, from Sanchez et al, 2016.

References:

Grattan-Bellew, P, A. Danay, *Comparison of laboratory and field evaluation of alkali-silica reaction in large dams,* International Conference on Concrete Alkali-Aggregate reactions in Hydroelectric Plants and Dams, Canadian Electrical Association, Oct 1992, 23p.

Grattan-Bellew, P., *Petrographic methods for distinguishing between alkali-silica, alkali-carbonate reactions and other mechanisms of concrete deterioration*, In Proceedings of the 14th International Conference on Alkali-Aggregate Reactivity in Concrete, Drimalas, T., Ideker J.H. and Fournier, B. Eds., Austin, Texas, May 2012, 10p.

Sanchez, L., Fournier, B., Jolin, M., and Duchesne, J., *"Reliable quantification of AAR damage through assessment of the Damage Rating Index (DRI),"* Cement and Concrete Research, Vol. 67, 2015, pp. 74-92.

Sanchez,L., B. Fournier, M. Jolin, J. Bastien, D. Mitchell, M. Noel, *Thorough characterization of concrete damage caused by AAR through the use of multi-level approach,* Proceedings of 15th International Conference on alkali-aggregate reaction, Sao Paolo, 2016. paper 25.

Sanchez,L., B. Fournier, M. Jolin, D. Mitchell, J. Bastien, *Overall assessment of Alkali-Aggregate Reaction (AAR) in concretes presenting different strengths and incorporating a wide range of reactive aggregate types and natures,* Cement and Concrete Research, 93, 17-31, 2017.