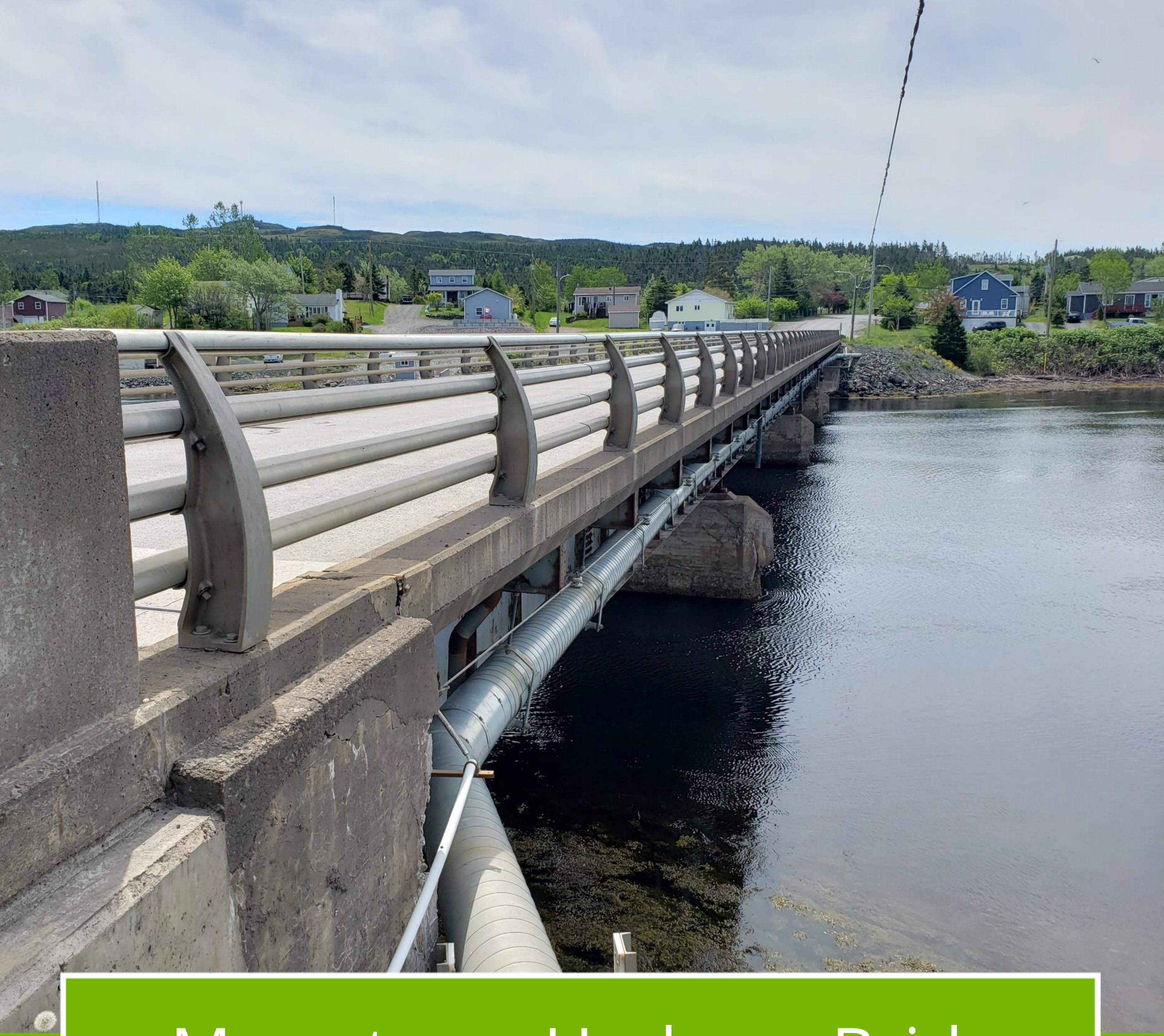

Marystown Harbour Bridge

Condition Assessment Report

223049.10 | April 2023

D	Issued for Final Report Rev 1	C. Jim	Apr 17, 2023	M. Warren
C	Issued for Final Report	C. Jim	Apr 4, 2023	M. Warren
B	Issued for Draft Rev 1	C. Jim	Mar 23, 2023	M. Warren
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Issue or Revision		**Reviewed By:**	**Date**	**Issued By:**

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April 17, 2023

Mike Button, P.Eng.
Highway Design and Construction
Department of Transportation and Infrastructure
Government of Newfoundland & Labrador

Dear Mr. Button:

RE: Marystown Harbour Bridge – Condition Assessment Report

CBCL Limited is please to submit this final report for the condition assessment of Marystown Harbour Bridge for your review and commentary.

Yours very truly,

CBCL Limited

Prepared by:	Reviewed by:
Mitchell Warren, B.Eng., EIT	Colin Jim, P.Eng.
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Project No: 223049.10

Contents

Appendices

Chapter 1 Project Background

The Newfoundland and Labrador Department of Transportation and Infrastructure (the Department) have procured CBCL Limited (CBCL) to assess the needs for extending the service life of Marystown Harbour Bridge (locally referred to as Canning's Bridge). The scope of work required CBCL to complete a condition assessment included a detailed inspection, material testing, evaluation, and rehabilitation/ replacement plan that will extend the service life of the bridge as efficiently and safely as possible.

Marystown Harbour Bridge is located on Route 220 in the town of Marystown NL. The bridge transverses the seawater narrows which connect Southwest Arm inlet to Placentia Bay (Figure 1-1). To support the Department's vision of safe, reliable, and sustainable infrastructure, it is imperative that this bridge is assessed and rehabilitated thoroughly to ensure the social and economic needs of the region are met.

This report has been prepared to provide the Department with the information required to make informed decisions on extending the life of this bridge in a cost-effective manner. The report provides a detailed assessment on the current physical condition of the bridge, an evaluation on the code permitted load carrying capacity of the bridge, and recommended rehabilitation options to repair, strengthen, and prolong the service life of the bridge and an alternative replacement option. The report concludes with a comparative analysis of rehabilitation versus replacement.

1.1 Project Objectives

The project objectives, as specified in the Request for Proposals for this project (RFP), include the following:

- Review previous inspection reports, site photos, and drawings of the bridge.
- Conduct a detailed physical condition assessment of the bridge including a hands-on site inspection accompanied by non-destructive and destructive material testing.
- Perform a structural engineering load evaluation in accordance with CAN/CSA S6-19 - Canadian Highway Bridge Design Code (CHBDC) including a live load analysis, seismic load analysis, fatigue analysis, and splice plate analysis.
- Develop a rehabilitation plan to extend the service life of this bridge to the year 2050.
- Develop replacement options for this bridge.
- Complete an evaluation of rehabilitation versus replacement.

- Complete a rehabilitation tender package (if required) and provide engineering support during construction.
- Additional scope of steel coupon testing was added following the completion of the preliminary evaluation.

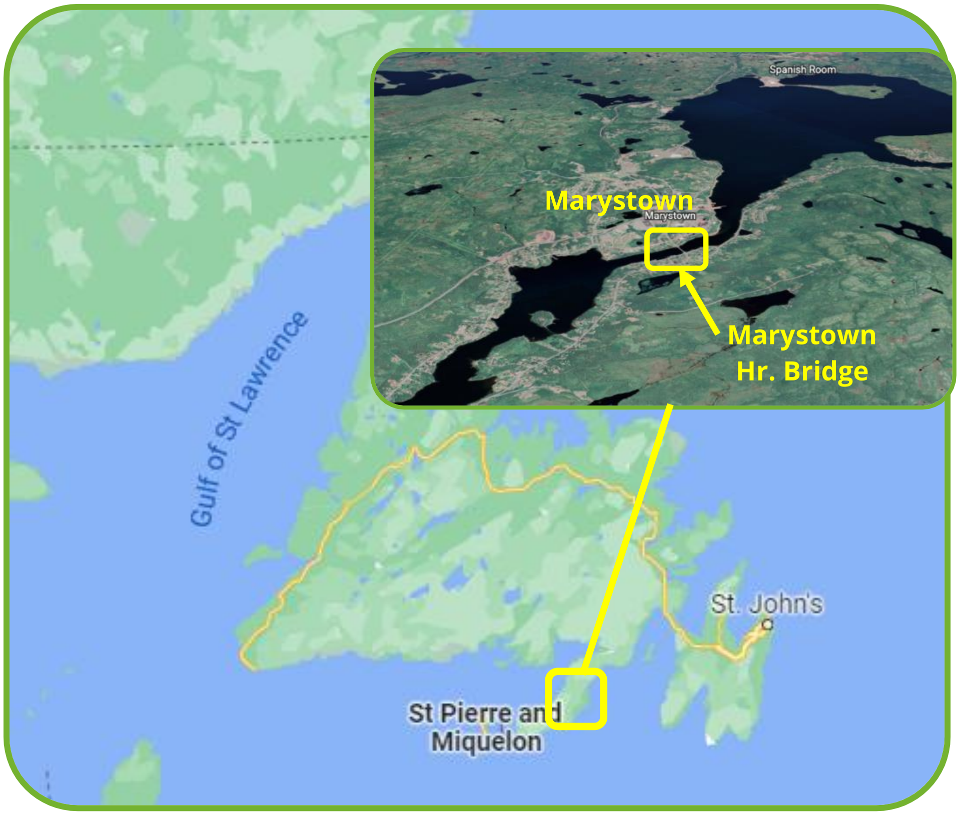

Figure 1-1: Marystown Harbour Bridge Location

Figure 1-2: Marystown Harbour Bridge

1.2 Structure Description

Marystown Harbour Bridge was constructed in 1957. The bridge consists of two structures: a swing span (that is currently fixed in place) and a continuous span. The swing span superstructure consists of a reinforced concrete deck supported on steel stringers, floor beams, and two tapered plate girders that span approximately 40 m (Figure 1-3). The continuous span superstructure consists of a reinforced concrete deck supported on steel floor beams, and two continuous plate girders that span a total length of approximately 80 m over two piers (Figure 1-4). The substructure for the bridge consists of reinforced concrete piers and abutments. The abutments and the northernmost pier (P1) are founded on piles driven to bedrock. The three other piers (P2, P3, and Pivot Pier) are cast on bedrock. The bridge has a 7.314 m clear roadway width carrying two lanes of traffic. The structure is built on a north-south straight alignment with a 0% grade. The existing structural drawings provided by the Department are appended to Appendix A.

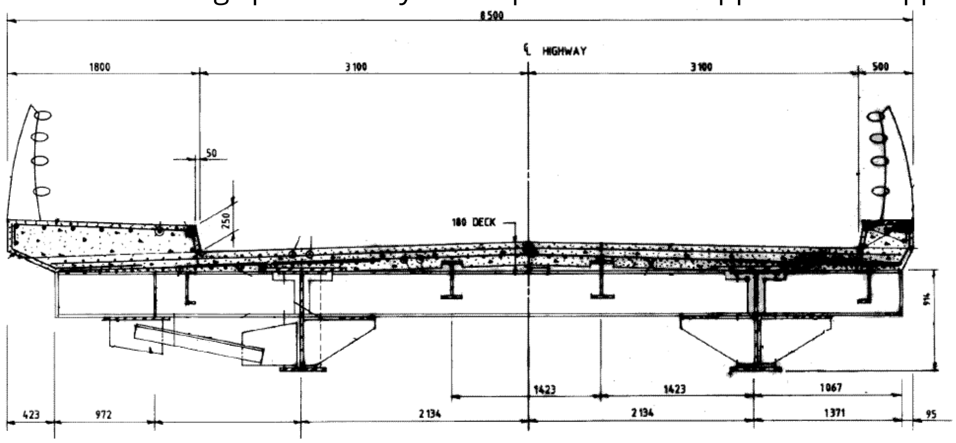

Figure 1-3: Swing Span Section

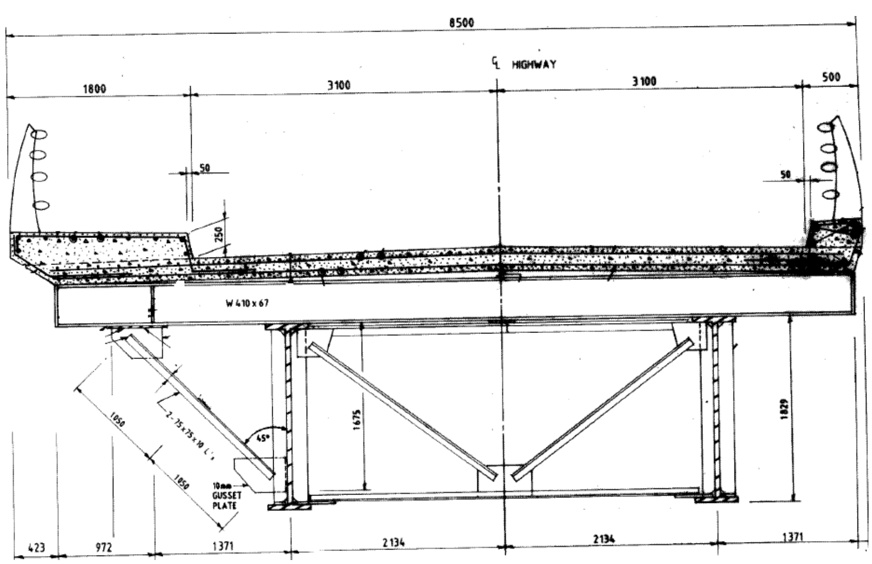

Figure 1-4: Continuous Span Section

Chapter 2 Inspection

CBCL performed a visual assessment of Marystown Harbour Bridge from September 26 to 30, 2022 with the aide of rope access technicians from Tacten Industrial Services (Tacten) and underwater inspection services from Sea-Force Diving Limited. This chapter provides a brief overview of the nomenclature used during the inspection, the inspection scope, rating methodology, and a summary of the major findings that were discovered.

Following the completion of the preliminary evaluation, CBCL returned to site in January 2023 to complete additional UT measurements and steel coupon testing as described in Section 3.4.

2.1 Nomenclature

The structural elements referenced in this report have been grouped and numbered in accordance with

Figure 2-1. The bridge is divided into two structures: continuous span and swing span. The continuous span is comprised of three spans of a continuous steel girder. The continuous span starts at the north abutment and spans in a southern direction over three piers: P1, P2, and P3. Girder G1 is the eastern girder and girder G2 is the western girder. Floor beams span transversely over the main girders. They are numbered sequentially as FB1, FB2, etc. starting from the North abutment. The girders are braced out-of-plane with Chevron style diaphragms. The diaphragms are numbered sequentially as D1, D2, etc. starting from the North abutment. The swing span was the original articulating portion of the bridge that rotated about the pivot pier. It is comprised of two spans: span 1 extends from P3 to the pivot pier and span 2 from the pivot pier to the south abutment. Swing span girder nomenclature is similar to the Continuous Span.

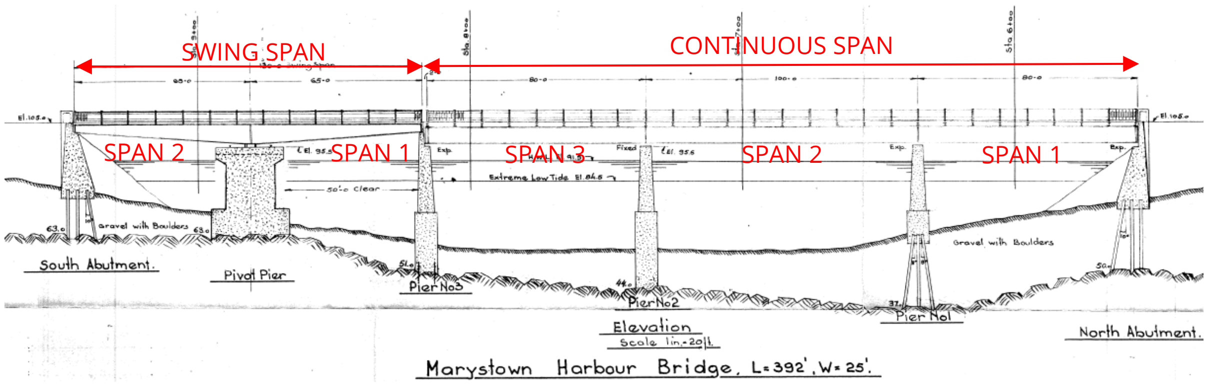

Figure 2-1: Marystown Harbour Bridge Profile

2.2 Inspection Scope

This bridge inspection required a detailed comprehensive visual assessment of the components of the sub- and super-structures to determine the extent, location, and severity of materials defects, and to identify any structural performance deficiencies. Every observed defect was measured, recorded, and photographed.

To acquire the above information, CBCL's scope of work also included:

- A review of the existing record drawings, inspection reports, and historic rehabilitation efforts.
- The provision of a rope access technician team to get an up-close visual assessment of all the components of the bridge and to perform magnetic particle testing, concrete coring, and ultrasonic thickness measurements.
- A dive inspection to evaluate the condition of the pier shafts and footings underwater.

The concrete components of the substructure were examined visually (within arms reach) and by hammer sounding. Rope access technicians were used, under the supervision of CBCL's inspectors, to access areas that were inaccessible from land. Concrete cores were extracted for testing from each bridge substructure component.

The two girders, floor beams, diaphragms, and bracing components that made up the superstructure were inspected visually by the rope access team. NDT technicians performed ultrasonic thickness measurements and magnetic particle testing (MPT) on the splice plates, as directed by CBCL.

2.3 Bridge Rating Methodology

This bridge inspection was conducted as an element-based inspection. The bridge components were divided into the following groups: abutments, approaches, barriers, beams, decks, joints, piers, and sidewalk/curbs. Each group was sub-divided into elements as provided in Table 2-1.

During the inspection, elements were reported in Ontario Structures Inspection Manual (OSIM) format and had their condition assigned as excellent, good, fair, or poor. In accordance with OSIM standards, all elements were assigned a suspected Performance Deficiency Code, a Maintenance Need Code, and a recommended repair solution based on the severity of the defects that were observed, if applicable.

Table 2-1: Bridge inspection elements.

Element Group	Element Name
Abutments	Abutment Walls
	Ballast Walls
	Wingwall
	Bearings
Approaches	Wearing Surface
	Barriers
Barriers	Railing System
	Posts
Beams	Girders
	Floor Beams
	Diaphragms
Bracing	Bracing (Sidewalk Strut)
	Bracing (Horizontal)
Coating	Structural Steel
Deck	Deck Top
	Soffit
	Drainage System
Joints	Armoring
	Seals and Sealants
Piers	Shaft
	Bearings
Sidewalk/curbs	Sidewalk
	Curb

2.3.1 Material Condition Rating

The Material Condition Rating system is used to rank an element based on the severity of the defects present. OSIM defines four Material Condition ratings: excellent, good, poor, and fair (Table 2-2). For each element, the inspector assesses and records the number of defects (area, length, or unit as appropriate) and the severity of the defects. The severity of defects is defined quantitatively in OSIM. Each element is assigned a material condition rating based on the inspector's assessment.

Table 2-2: Material Condition Rating

Rating	Description
Excellent	Element is in new (as constructed) condition. No visible deterioration type defects are present and remedial action is not required.
Good	Elements which experience 'light' defects. These types of defects would not normally trigger any remedial action since the overall performance of the element is not affected.
Fair	Elements which experience 'medium' defects. These types of defects may trigger a 'preventative maintenance' type of remedial action.
Poor	Elements with severe to very severe defects. These types of defects would normally trigger rehabilitation or replacement if the extent and location affect overall performance of the element.

2.3.2 Suspected Performance Deficiencies

An element is assigned a suspected performance deficiency when its ability to perform its intended function is in question. The performance deficiency is selected from a standardized list as published in the Ontario Structures Information Manual (OSIM). If the inspector does not suspect a performance deficiency is associated with an element, then a performance deficiency of 00 is assigned to that element. The standard list of deficiencies is shown below in Table 2-3.

Table 2-3: Standard list of suspected performance deficiencies

00	None	**06**	Bearings not uniformly loaded/unstable	**12**	Slippery Surfaces
01	Load carrying capacity	**07**	Jammed expansion joint	**13**	Flooding/channel blockage
02	Excessive Deformation (Deflections and Rotations)	**08**	Pedestrian/vehicular hazard	**14**	Undermining of foundations
03	Continuing settlement	**09**	Rough Riding Surface	**15**	Unstable embankments
04	Continuing Movements	**10**	Surface Ponding	**16**	Other
05	Seized Bearings	**11**	Deck Drainage		

2.3.3 Maintenance Needs

If the inspector deems that an element requires a maintenance need, then they select a code from an OSIM standard list of Maintenance Needs (Table 2-4). This code provides the reviewer and client with an indication of the nature of that defect. The assigned maintenance need is not necessarily descriptive of the rehabilitation required as it considers maintenance at the element level and not an overall structure level.

Table 2-4: Standard List of Maintenance Needs

01	Lift and Swing Bridge Maintenance	**07**	Repair to structural steel	**13**	Erosion Control at Bridges
02	Bridge Cleaning	**08**	Repair of bridge concrete	**14**	Concrete Sealing
03	Bridge Handrail Maintenance	**09**	Repair of bridge timber	**15**	Rout and Seal
04	Painting steel bridge structures	**10**	Bailey Bridge Maintenance	**16**	Bridge Deck Drainage
05	Bridge deck joint repair	**11**	Animals/Pest Control	**17**	Scaling (loose concrete or ACR Steel)
06	Bridge Bearing Maintenance	**12**	Bridge Surface Repair	**18**	Other

2.3.4 Recommended Work

Within the inspection data forms there is a section called "Recommended Work". The recommendations within these forms are provided at the element level and from the perspective of the inspector rather than at the bridge level by the rehabilitation designer. These recommendations are reviewed by the rehabilitation designer, who then takes a more holistic approach to the ultimate rehabilitation plan. A proposed rehabilitation plan will be presented in the final report.

2.4 Summary of Major Inspection Findings

CBCL's detailed inspection forms are provided in Appendix B with photos in Appendix C. The deficiencies found during the inspection are illustrated in sketches in Appendix D.

This section presents a tabulated summary of the major findings of the inspection with a focus on those elements with poor condition states, suspected performance deficiencies, and elements deemed to have critical importance to the structure and the rehabilitation initiative. The sub-sections of this section are broken down at the element group level.

2.4.1 Abutments

The following table summarizes the major defects found and the recommended repairs for the abutments.

Table 2-5: Summary of major defects and recommended repairs to the abutments

Element	Location	Condition State (m^2)			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Abutment Walls	North	16.7	1.8	12	00	08	Very severe delamination and spalling are present on both abutments. To preserve the life of the structure and mitigate future deterioration, all unsound concrete should be removed and encapsulated.
	South	31.3	2.2	14.5			
Ballast Walls	North	14.3	1	6.8	00	08	Very severe delamination and map cracking is present. To preserve the life of the structure and mitigate future deterioration, all unsound concrete should be removed, and the ballast walls should be encapsulated with a layer of reinforced concrete.
	South	9	0.2	1			
Wingwall	SW	25.4	0.2	1.4	00	08	Map cracking is present throughout all surfaces. The unsound concrete should be removed the walls should be encapsulated with a layer on reinforced concrete.
	NE	9.2	0.2	1			
	SE	0	0	8.6			

2.4.2 Bearings

The following table summarizes the major defects found and the recommended repairs for the bearings. The findings were supplemented by the rope access visual inspection results contained in Appendix E.

Table 2-6: Summary of major defects and recommended repairs to the piers

Element	Location	Condition State (Each)			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Bearing	P1	0	0	2	05	06	Severe corrosion is present on all bearings and anchor rods. It is suspected, based on the material condition, that the bearings are not functioning as intended. CBCL recommends removing and replacing all bearings.
	P2	0	0	2			
	P3	0	0	4			
	Pivot Pier	0	0	1			
	South Abutment	0	0	2			
	North Abutment	0	0	2			

2.4.3 Approaches

The following table summarizes the major defects found and the recommended repairs to the approaches.

Table 2-7: Summary of major defects and recommended repairs to the approaches

Element	Location	Condition State (m^2) Good	Fair	Poor	Performance Deficiency	Maintenance Needs	Major Findings & Work Required
Wearing surface	North	0	28	3	09	12	The asphalt patches on the approach are uneven and do not provide a smooth transition to the bridge deck. Some vehicles were witnessed to slow down when entering the bridge which caused trailing vehicles to brake suddenly to avoid a collision. It is recommended that the asphalt be replaced as soon as possible.
	South	29	0	2			

2.4.4 Barriers

The following table summarizes the major defects found and the recommended repairs for the bridge traffic barriers.

Table 2-8: Summary of major deck defects and recommended work for the barriers

Element	Location	Condition State Good	Fair	Poor	Performance Deficiency	Maintenance Needs	Major Findings & Work Required
Railing System (m)	East & West	40	180	20	01	00	This railing does not meet TL4 requirements. This barrier should be replaced with an approved traffic barrier system.
Posts (Each)	East & West	40	0	1	08	00	Slight deficiency present on 1 post. The posts should be replaced with an approved traffic barrier system.

2.4.5 Beams & Bracing

In general, the steel structural elements were found to be in poor condition. There were no permanent deformations observed in the main girder or beam elements however there was significant corrosion loss to the diaphragm steel, top and bottom flanges of the main girders, and the nuts and bolts of all splice plate connections. The results of the inspection have assigned a performance deficiency rating to some elements.

The majority of the steel inspection was performed by Tacten rope access under CBCL's supervision. A copy of their visual report is provided in Appendix E. The amount of corrosion present on the steel was delineated with ultrasonic thickness measurements completed by Tacten (Appendix F).

Table 2-9: Summary of major coating defects and recommended work for the beam elements

Element	Location	Condition State			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Girders (m^2)	Cont. Span	0	368	420	01	07	Medium corrosion is present on the girder bottom and top flanges. There is severe corrosion on the bolts at each splice plate. It is recommended to reinforce the top and bottom chords with plates to restore load capacity and to replace the bolts in each splice plate.
Girders (m^2)	Swing Span 1 & 2	0	200	160	01	07	Medium corrosion is present on the girders bottom and top flanges. There is severe corrosion on the bolts at each connection plate. It is recommended to reinforce the top and bottom chords with plates to restore load capacity and to replace the bolts in each splice plate.
Diaphragms (Each)	Cont. Span 1,2, & 3	0	0	10	01	07	Very severe corrosion in all members. Remove and replace all members
Bracing (Each)	Bracing (Horizontal)	0	1	21	01	07	Very severe corrosion in all members. Remove and replace all members

2.4.6 Coatings

From general observations the coating appeared to be in poor condition. An on-site coatings assessment was performed by Tacten and the results of their assessment are summarized in the following table and available in Appendix G.

Table 2-10: Summary of major coating defects and recommended work

Element	Location	Condition State (m^2)			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Structural Steel	Continuous & Swing Span	0	419	1530	00	04	The paint coating is in an advanced stage of deterioration on all members and no longer offers any corrosion protection of the steel from corrosion. It is recommended that the bridge steel members be sandblasted and re-coated.

2.4.7 Deck

The following table summarizes the major defects found and the recommended repairs for the bridge deck.

Table 2-11: Summary of major deck defects and recommended work for the bridge deck

Element	Location	Condition State (m^2)			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Deck Top	Cont. Spans 1, 2, 3	670	1	8	00	08	There is approx. 8 m^2 of spalled/delaminated concrete that should be repaired to mitigate further deterioration.
Deck Top	Swing Spans 1 & 2	337	1	4	00	08	There is approx. 4 m^2 of spalled/delaminated concrete that should be repaired to mitigate further deterioration.
Soffit	Swing Span 1&2	868	0.15	1	00	08	There is approx. 0.6 m^2 of spalled/delaminated concrete near the South abutment that should be repaired to mitigate further deterioration.

2.4.8 Piers

The following table summarizes the major defects found and the recommended repairs for the piers. The findings were supplemented by the diving inspection results contained in Appendix H.

Table 2-12: Summary of major defects and recommended repairs to the piers

Element	Location	Condition State (m^2)			Performance Deficiency	Maintenance Needs	Major Findings & Work Required
		Good	Fair	Poor			
Pier Shaft	P1	78	85	50	00	08	The concrete walls have been very severely eroded. Large areas of very severe delamination and spalling are present on all sides. Each pier has a wide vertical crack in the middle of the wall that extends from the top to the foundation. The piers should be encapsulated or replaced. The final recommendation will rely on the concrete test results and the feasibility of a remedial or replacement option.
	P2	75	75	37			
	P3	73	100	32			
Pier Shaft	Pivot Pier	12	150	88	00	08	Very severe delamination is present on 50% of the entire surface with wide map cracking and efflorescence. Undermining is also present on the north side of the footing. The pier should be encapsulated in a layer of new concrete with GFRP shrinkage/temperature reinforcing. This purpose of this new layer is to protect the substructure from further chloride ingress, freezing and thawing effects, and ice abrasion. The void under the footing should be filled with tremie concrete and scour protection be put in place on upstream and downstream sides.

Chapter 3 Materials Testing

The materials testing program completed for this bridge consisted of a concrete coring and testing program completed by CBCL, magnetic particle testing completed by Tacten, a hazardous materials assessment completed by ALL-TECH, and steel coupon testing completed by AMC. Separate reports for each of these tests are provided in Appendices I, J, K, and L respectively. The interpreted results of these reports, as they affect the existing bridge condition and rehabilitation decision-making are discussed below.

3.1 Concrete Testing

CBCL performed laboratory materials testing on the concrete core samples taken from elements of the Marystown Harbour bridge to quantitatively assess the concrete material condition. Based on our field observations and measurements, the laboratory testing results, and our professional analysis and opinion, we have provided general comments about the concrete material integrity, the need for repairs, and the estimated expected remaining service life.

3.1.1 Concrete Sampling

In total thirty-four (34) 4” nominal diameter cores were removed from various elements of Marystown Harbour bridge to evaluate the condition of the concrete. Coring locations were selected by CBCL with the intent of obtaining samples from a broad distribution of location and exposure conditions. Ten (10) cores were removed from the bridge deck, five (5) from the south abutment, six (6) from the north abutment, five (5) from Pier 1, two (2) from Pier 2, three (3) from Pier 3 and three (3) from the swing pivot pier. Core sampling was performed by CBCL staff in accordance with CSA A23.2-14C: Obtaining and testing drilled cores for compressive strength testing. Following removal, all cores were transported to CBCL’s CCIL certified laboratory in Saint John, New Brunswick for visual examination and testing. Testing included compressive strength, air void analysis, chloride ion content, petrographic examination, and carbonation testing. Table 3-1 presents a list of all cores, the element in which they were removed and additional location details if available.

Table 3-1: Concrete Core Sampling Locations

Core ID	Bridge Element	Other Location Details
D1	Deck	Between North Abutment and Pier 1
D2	Deck	Between Pier 1 and Pier 2
D3	Deck	Between Pier 2 and Pier 3
D4	Deck	Between Pier 1 and Pier 2
D5	Deck	Between Pier 2 and Pier 3
D6	Deck	Between Pier 3 and Pivot Pier
D7	Deck	Between Pier 3 and Pivot Pier
D8	Deck	Between Pivot Pier and South Abutment
D9	Deck	Between Pivot Pier and South Abutment
D10	Deck	Between Pier 3 and Pivot Pier
9	South Abutment	Back wall
11	South Abutment	East wing wall
12	South Abutment	Beam seat
13	South Abutment	Foundation. Approximately middle
14	South Abutment	Foundation. Near ground level
15	North Abutment	Back wall
16	North Abutment	Back wall. Approximately middle
17	North Abutment	West wing wall
18	North Abutment	Top face of bearing seat (East end)
19	North Abutment	Top face of bearing seat (Approximately middle)
20	North Abutment	West half of foundation
21	Pier 2	Above high-water line
22	Pier 2	Top face
23	Pier 3	Top face
24	Pier 3	Above high-water line
25	Pier 3	Below high-water line in tidal zone
26	Pier 1	Below high-water line in tidal zone
27	Pier 1	Above high-water line
28	Pier 1	Above high-water line
29	Pier 1	Below high-water line in tidal zone
30	Pier 1	Top face
31	Pivot Pier	Top face
32	Pivot Pier	Top face
33	Pivot Pier	Vertical face

3.1.2 Visual Examination of Cores

All thirty-four (34) cores were visually inspected for cracking, aggregate size/quality and overall appearance prior to measuring mechanical and durability properties. Cores removed from the bridge deck (cores D1-D10) were observed to be in overall fair to poor condition. Of the ten (10) cores removed, two (D6 and D10) were mostly rubble while the remaining eight were in fair and sound condition. Core D1, was removed in two pieces likely due to delamination cracking at a depth at approximately 50 mm below the deck surface. A nominal maximum size aggregate of 14 mm was found in all bridge deck cores. The aggregate was deemed to be in fair condition with sporadic darkened rims indicative of alkali-silica reactivity (ASR). Significant, large voids, indicative of poor concrete consolidation at the time of placement were observed in two (2) cores while minor or small areas of entrapped air were observed in five (5) cores. Cores removed from the north and south abutments (9, 16, 17, 18, 19, 20 and 11, 12, 13, 14, respectively) were also in poor condition. Of the ten (10) cores removed, six (6) cores were removed in sections of rubble due to large cracks parallel to the concrete surface ranging in depth of 50 to 200 mm. Nominal maximum size aggregates of 20, 40 and 60 mm were found in concrete regardless of abutment. Cores removed from Pier 1, Pier 2, Pier 3, and the pivot pier were deemed to be in fair to poor condition. Of the thirteen (13) cores (cores 21, 22, 23, 24, 25, 26, 27, 28, 29, 30, 31, 32, 33), only one core (core 28) was removed in multiple sections likely due to a delaminated crack at a depth of approximately 200 mm. Although the remaining cores were removed in one piece, almost all cores displayed weak interfacial transition zones between the aggregate and paste and significant cracking within the aggregate and paste which is assumed to be indicative of alkali silica reaction. Nominal maximum size aggregates ranging from 20 to 40 mm were found in all cores. All core photographs are presented in Appendix I.

3.1.3 Laboratory Testing

A select number of cores were selected for laboratory testing which included compressive strength, chloride ion content, hardened air void analysis, petrographic examination, and carbonation measurements. The results are presented in the following sections.

3.1.3.1 Compressive Strength

Unconfined compressive strength testing in accordance with CSA A23.2:19-14C *Obtaining and testing drilled cores for compressive strength*, was performed on eight (8) core samples from the bridge deck, North Abutment, Pier 1, Pier 2, and Pier 3. Compressive strength results from the various bridge elements are presented in Table 3-2. Compressive strength results from the North Abutment are significantly lower than other bridge elements. The compressive strength of concrete recommended for use in a structure that is structurally reinforced and exposed to chlorides, such as this bridge, would be a minimum of 35 MPa (CSA A23.1 Class C1) from a materials durability perspective.

Table 3-2: Compressive Strength Results

Bridge Element	Compressive Strength (MPa)		No. of Cores Tested
	Range	Average	
North Abutment	34.8	34.8	1
Pier 1	41.3 – 47.2	43.3	3
Pier 2	44.9	44.9	1
Pier 3	46.0 – 50.7	48.3	2
Deck	43.0	43.0	1

3.1.3.2 Acid-Soluble Chlorides

The concentration of chlorides relative to depth was measured in accordance with ASTM C1152-20: *Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete*, on twelve (12) cores from various elements. Due to the range of concrete cover, cores from the bridge deck (Cores D2, D4, D7, D9) were tested for chloride ion content at 10 mm increments while all other cores were tested for chloride concentration at 15 mm increments. Table 3-3 and Table 3-4 present acid soluble chloride ion concentrations relative to depth for concrete from bridge deck and other elements, respectively. Acid-soluble chloride ion concentrations above the threshold at which corrosion of black steel may be initiated are presented in **red.** All results were corrected using a typically used background chloride ion concentration of 0.02% by mass of concrete.

Table 3-3: Chloride Ion Content (% by mass concrete) of Bridge Deck Cores

Mid Depth (mm)	Core D2	Core D4	Core D7	Core D9
10	0.442	0.513	0.478	0.398
20	0.348	0.429	0.381	0.305
30	0.308	0.315	0.253	0.218
40	0.252	0.201	0.214	0.164
50	0.179	0.105	0.132	0.079

Note: Chloride ion contents adjusted assuming a commonly used background chloride ion concentration of 0.02% by mass of concrete.

Table 3-4: Chloride Ion Content (% by mass concrete) of all other elements

Mid Depth (mm)	Core 9	Core 12	Core 15	Core 22	Core 23	Core 25	Core 27	Core 30
15	0.167	0.186	0.141	0.151	0.083	0.513	0.156	0.392
30	0.134	0.160	0.100	0.195	0.088	0.203	0.099	0.326
45	0.143	0.139	0.145	0.150	0.085	0.210	0.102	0.222
60	0.158	0.145	0.100	0.090	0.086	0.225	0.096	0.202
75	0.158	0.118	0.154	0.092	0.062	0.210	0.092	0.228

Note: Chloride ion contents adjusted assuming a commonly used background chloride ion concentration of 0.02% by mass of concrete.

Regardless of location, all cores present a chloride concentration exceeding a concentration of 0.05% by mass of concrete at the maximum depth tested. A chloride threshold of 0.05% by mass of concrete is the typically used threshold needed to initiate corrosion (assuming moisture and oxygen are present at the surface of the steel). As per design drawings of the concrete deck, the top map of reinforcing steel is found 50 mm below the top of the deck surface and assumed to be at 75 mm in the abutments. Regardless of location, it is very likely that corrosion has initiated at these locations. Figure 3-1 presents a graphical representation of chloride concentration relative to mid-depth of all cores. The decreased surface concentration in cores removed from the piers and abutments (Cores 9, 12, 15, 22, 23, 27, 30) is likely due to factors including exposure conditions, cement content, chloride washout, binding capacity, surface scaling, and skin effect. Regardless of surface concentration, a concentration in the range of 0.079-0.179% at a mid depth of 50 mm is present in deck concrete whereas a concentration in the range of 0.062-0.228% at a depth of 75 mm is observed in all other concrete elements.

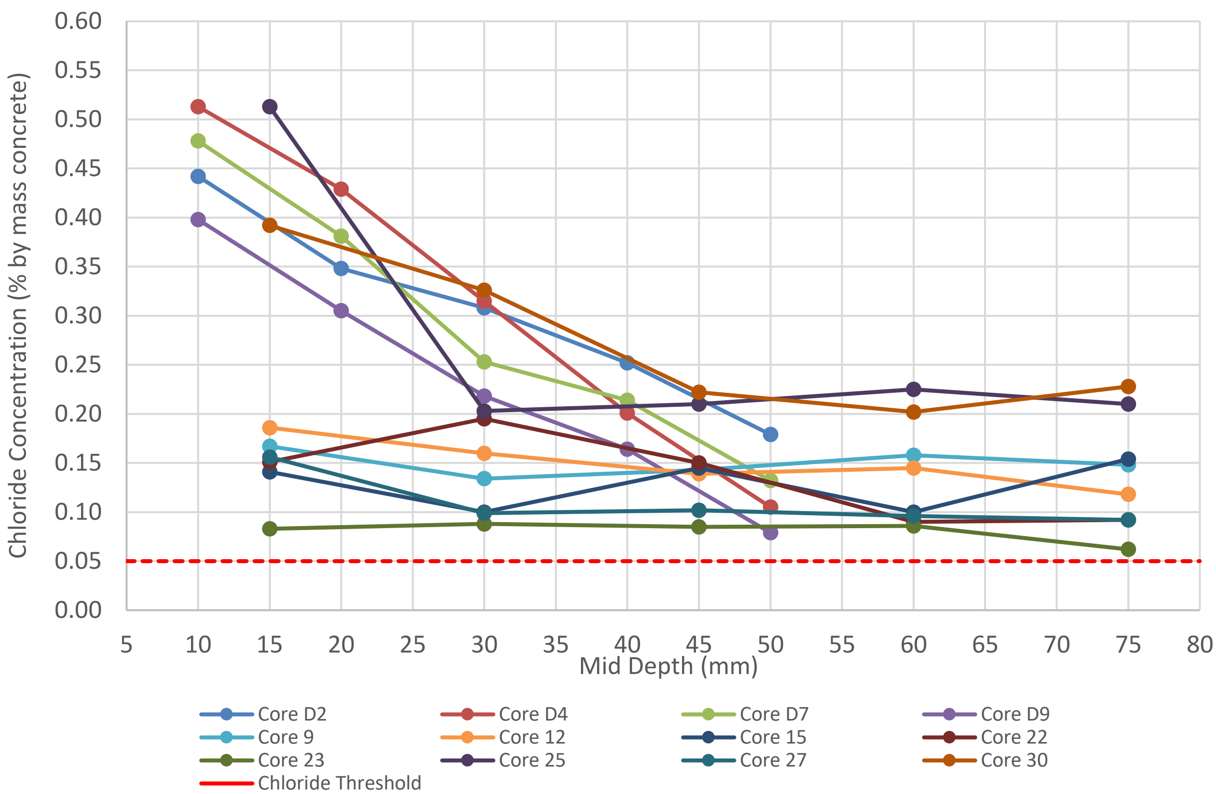

Figure 3-1: Chloride penetration relative to depth for all cores

3.1.3.3 Carbonation

Carbonation is a chemical reaction that occurs when atmospheric carbon dioxide penetrates a Portland cement concrete and dissolves in pore water, creating carbonic acid, which then reacts with calcium hydroxide and produces calcium carbonate. This reaction results in a reduction in the concrete pH from highly alkaline to approaching neutral as well as an increase in porosity in the affected concrete. At elevated levels of alkalinity like those normally found in uncarbonated concrete, a passive layer forms around the reinforcing steel which protects it from corrosive action. When the pH of the concrete significantly drops however, as is the case in concrete that has been affected by carbonation, the passive layer disappears, and corrosive action may occur.

Carbonation depth measurements were carried out on five (5) cores from various elements of the bridge concrete. Table 3-5 presents average carbonation depths. Carbonation measurements were performed by splitting cores tangentially and spraying the freshly fractured surface with a 1% phenolphthalein solution, which is 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 < 0.2). At pH levels less than 9.2 there is a concern as at this level, the ferric oxide layer used to protect reinforcing steel from corrosion is unstable. In the presence of oxygen and moisture, corrosion is likely to initiate. Carbonation induced corrosion occurs at an optimum relative humidity of 45-65% and results in a decrease in pH and eventually the initiation of corrosion in the event the carbonation depth reaches the depth of reinforcing steel. Carbonation is of concern because, (i) corrosion of steel may be initiated once the carbonation front reaches the surface of steel; and (ii) carbonation may render the near surface concrete less resistant to abrasion, salt scaling and chloride ingress.

Based on the result presented in Table 3-5, the depth of carbonation does not exceed 24 mm, which based on design drawings has yet to reach the surface of reinforcing steel. While carbonation induced corrosion is likely not of concern, the presence of carbonation has likely contributed to surface abrasion, scaling, and chloride ingress. Photographs of carbonation specimens are presented in Appendix I.

Table 3-5: Approximate Depth of Carbonation (mm)

Core ID	Bridge Element	Approximate Depth (mm)
D5	Deck	2
D8	Deck	10
11	South Abutment	4-7
28	Pier 1	6
32	Swing Pier	14-24

3.1.3.4 Alkali-Silica Reactivity

Visual inspection of the concrete core samples indicated that there was evidence that alkali-silica reaction (ASR) had occurred. To confirm alkali-silica reaction product was present, Cornell gel fluorescence testing was performed on a selection of the core sample fragments in accordance with standard test method *AASHTO T299: Rapid Identification of Alkali-Silica Reaction Products in Concrete*. Evidence of ASR was not identified on core samples taken from the bridge deck; gel fluorescence testing was not performed on bridge deck samples.

Fragments from five (5) core samples were tested. All five (5) sample fragments fluoresced under UV light after being exposed to a uranyl acetate solution, indicating the likely presence of alkali-silica reaction product. Images of the core samples in regular light, under UV light prior to being exposed to uranyl acetate, and under UV light following exposure to uranyl acetate are enclosed in Appendix I. Based on the intensity of the observed fluorescence, the locations of the observed fluorescence, and a visual examination of the core samples in regular light, we are of the opinion that the concrete represented by the core samples has undergone expansion due to alkali-silica reaction.

It should be noted that concrete that has undergone carbonation or concrete mixes containing fly ash and/or silica fume may fluoresce under UV light after being exposed to uranyl acetate. The fluorescence caused by carbonation, fly ash, or silica fume is typically observed to be relatively evenly distributed in the cement paste whereas fluorescence cause by ASR product is more concentrated around affected aggregate particles.

3.1.3.5 Air Void Parameters

Air void parameter measurements in accordance with ASTM C457-16*: Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete* were carried out on four (4) cores samples from the bridge deck, north abutment, pier 1 and pier 3 (cores D3, 16, 24, 29, respectively). The analyses determined that the total air content of the core samples ranged from 2.1% to 6.6%. The spacing factors were measured to range from 0.120 to 1.050 mm. The measured air content and spacing factor results are presented in Table 3-6. As per CSA A23.1:19 Clause 4.3.3. concrete exposed to cyclic freezing and thawing should have an average spacing factor less than 0.230mm with no single test results exceeding 0.260mm and a hardened air content greater than 3.0%. Two (2) of the four (4) cores tested have inadequate air contents and three (3) cores have spacing factors exceeding the average spacing factor requirements. The test results suggest that substructure elements (north abutment, pier 1 and pier 2) under saturated condition and exposed to cyclic freezing and thawing will deteriorate at an accelerated rate.

Table 3-6: Air Void Parameter Results

Core ID	Bridge Element	Air Content (%)	Spacing Factor (mm)
D3	Deck	6.6	0.120
16	North Abutment	2.1	0.370
24	Pier 3	2.4	1.050
29	Pier 1	3.5	0.550
Recommended Values (CSA A23.1:19, Clause 4.3.3)		≥ 3.0	≤ 0.260

3.1.3.6 Petrographic Examination & Damage Condition Rating

One (1) concrete core sample (Core 20) was examined under Petrographic examination and damage rating index (DRI) in accordance with standard test method *ASTM C856 - Standard Practice for Petrographic Examination of Hardened Concrete.*

As per the petrographic and DRI report presented in Appendix I the concrete is well proportioned, well consolidated and of moderate strength, but non air-entrained and showing strong evidence of alkali silica reaction (ASR). The presence of thaumasite was also found which is responsible for damaging sulphate attack and found in concrete exposed to moisture at relatively low temperatures.

The concrete has a damage rating index (DRI) of 431 which indicates that the concrete has been damaged by alkali-silica reaction. The amount of ASR product observed was judged to be moderate. The rock types present in the concrete are volcanic tuffs which are well known to be responsible for damaging ASR in Newfoundland.

3.1.4 Discussion and Recommendations

3.1.4.1 Bridge Substructure

Overall, concrete in the substructure elements were found to be in very poor condition. Concrete compressive strengths measured from cores taken from the piers exceeded that of CSA A23.1 Exposure Class C1 concrete, which is typically used under such an environment, however, cores from the North Abutment failed this requirement. The chloride ion concentrations in the substructure concrete at the depth of embedded steel present chloride concentrations greater than the threshold required to initiate and sustain corrosion (assuming oxygen and moisture are presented). Therefore, chloride induced corrosion is likely present at these locations to a depth of at least 75 mm. All cores were found to be insufficiently air entrained to resist cyclic freezing and thawing under saturated conditions. As a result, all substructure elements have sustained significant mass loss likely due to a combination of deterioration mechanisms including cyclic freezing and thawing. Evidence of alkali-silica reactivity was observed by both visual inspection and confirmed with Cornell gel fluorescence testing. The presence of ASR cracking has likely led to a pathway for chlorides and the initiation of chloride induced corrosion and cyclic freezing and thawing. While it is unknown which deterioration mechanisms initiated the deterioration of these elements, it is likely that ASR resulted in initial cracking which led to the propagation of moisture and subsequent chloride induced corrosion and cyclic freezing and thawing. As a result, significant section loss has occurred in all piers.

3.1.4.2 Bridge Deck

Overall, concrete in the bridge deck was found to be in poor to fair condition. The compressive strength of the bridge deck concrete was measured to be suitable for use in such an environment. The bridge deck concrete was measured to be sufficiently air entrained. Similarly, to all substructure elements, the chloride ion concentrations in the bridge deck concrete at the depth of embedded steel present chloride concentrations greater than the threshold required to initiate and sustain corrosion (assuming oxygen and moisture are presented). Evidence of alkali-silica reactivity was not observed during visual inspection of the deck or from inspecting core samples. Since the deck was replaced in 1992, it is assumed that a non-reactive aggregate was used.

3.1.4.3 Limitations of Materials Investigation

The results presented herein present results from samples collected and may or may not represent other elements on the bridge. It is however, CBCL's assumption that other areas of bridge elements not sampled are deteriorating at the same rate and under the same conditions as those cores investigated above.

Core samples were not drilled from sections of the pier elements below the low-tide level. It would be expected that the severity of the concrete defects in this region be less severe than in sections of the piers within the tidal zone or above high water as concrete below low water is not exposed to freezing and thawing and presence of oxygen is less to initiate chloride induced corrosion. Cycles of freezing and thawing, combined with ASR and corrosion of embedded reinforcing steel, are expected to have significantly contributed to the rate of concrete pier deterioration.

Ultrasonic pulse velocity (UPV) measurements were performed on elements of the bridge substructure. Due to the presence of significant cracking of the substructure elements however, repeatable UPV measurements were unable to be achieved. Correlation of the collected UPV data with laboratory-measured concrete core compressive strength test results was therefore unable to be successfully completed. As all compressive strengths measured from substructure core samples were found to be substandard, this lack of additional data is considered inconsequential.

Due to the presence of excessive cracking in the substructure elements and of saturated concrete conditions in the bridge deck, the ground penetrating radar (GPR) data that was collected during the field work was found to be slightly distorted. The reinforcing steel cover measurements used for chloride-ion profiling were therefore determined using a combination of the GPR data and cover measurements specified in the as built drawings. As the chloride ion concentrations measured in the substructure cores were found to be significantly above the corrosion threshold, it is believed that slight errors in assumed cover depths would have been inconsequential to the development of our conclusions on the concrete materials conditions.

3.2 Magnetic Particle Testing

Magnetic particle testing (MPT) was conducted by Tacten on a 25% representative sample of splice plates on the main girders. Three splice plates out of twelve were tested on the continuous span and one splice plate out of four was tested on the swing span (Figure 3-2). No relevant cracking was observed at the time of the examination. Tacten's MPT report is appended in Appendix J.

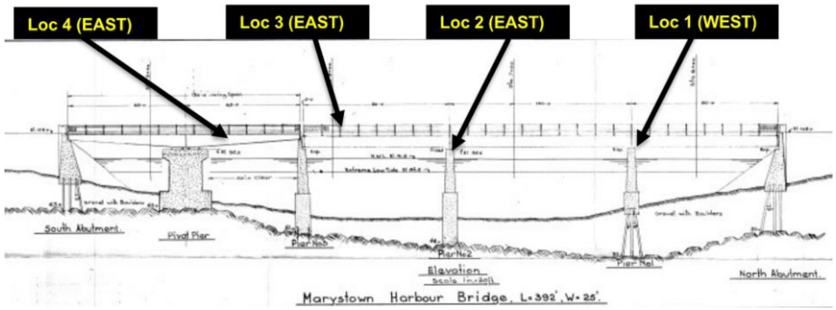

Figure 3-2: MPT Test Locations

3.3 Hazardous Material Testing

Paint samples were collected from the main girders and floor beams by CBCL and sent to ALL-TECH Environmental Services Limited for evaluation of arsenic, lead, and mercury concentration. The laboratory analyses found that lead was present at concentrations much higher than provincial guidelines for safe working limits, no arsenic was detected, and trace amounts of mercury was present but below provincial guidelines (Table 3-7). ALL-TECH recommends that all paint should be treated and disposed of as a hazardous waste (Appendix K).

Table 3-7: Summary of hazardous material testing results

Sample ID	Arsenic Content		Lead Content		Mercury Content	
	Concentration (mg/kg)	Guidelines (mg/kg)	Concentration (mg/kg)	Provincial Guidelines (mg/kg)	Concentration (mg/kg)	Provincial Guidelines (mg/kg)
NL10455-01	None Detected	12	9,100	600	0.068	10
NL10455-02	None Detected	12	19,000	600	0.088	10
NL10455-03	None Detected	12	90,000	600	None Detected	10
NL10455-04	None Detected	12	69,000	600	0.057	10

3.4 Steel Coupon Testing

The preliminary evaluation was completed utilizing the steel grades indicated on the original drawings as discussed in Section 4.2.2.1 (220 / 230 MPa). Following the completion of the preliminary evaluation, and in consultation with the Department, it was determined to extract steel coupons in an effort to increase the permitted load carrying capacity of the bridge. From January 9-12, 2023, Tacten extracted ten (10) coupons from the East and West girders of the continuous span under CBCL's direction (Figure 3-2). The coupons were sent to Atlantic Metallurgical Consulting Limited to test the coupons in accordance with CSA G40.20/G40.21 (Appendix L). In compliance with Clause A14.1.1 of the CHBDC, the yield strength to be utilized for the evaluation of the girders was determined to be 156 MPa.

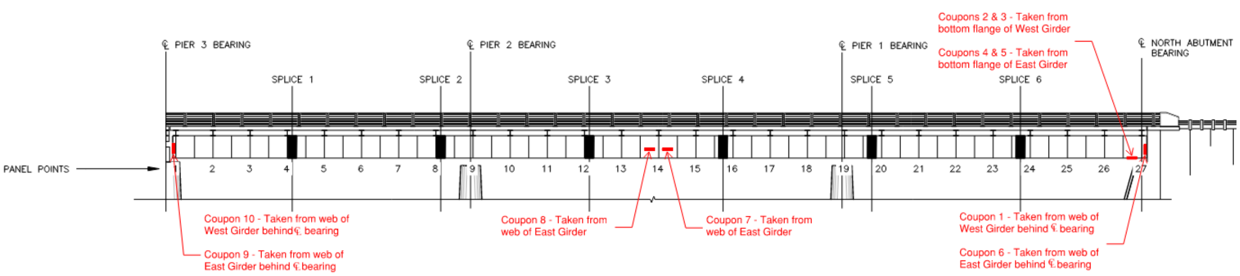

Figure 3-3: Steel Coupon Testing Locations

Chapter 4 Live Load Evaluation

Marystown Harbour Bridge was evaluated in accordance with Section 14 of the CHBDC. This chapter summarizes the scope of work, evaluation procedure, loading, and a summary of the results that were yielded from the load evaluation.

4.1 Scope of Work & Limitations

The scope of work for this section was to evaluate the structural live load carrying capacity of Marystown Harbour Bridge to identify any issues that may compromise unrestricted traffic flow on the bridge. This exercise included the following:

- A review of existing drawings and documentation.
- Confirmation of the structural member condition and geometry during the site inspection.
- A structural evaluation at Ultimate Limits State (ULS) in accordance with Section 14 CHBDC which included dead load, superimposed dead load, and live load. Resistances were calculated for all primary members as per CHBDC considering the original and current condition (including section loss).
- Preparation of a report which will include a clear discussion and justification for all assumptions, methodology, results, and a detailed list of recommendations to address all identified issues.

The following were not considered in this evaluation:

- Wind effects on the bridge.
- Analysis of bearings:
  - For analysis purposes, the bearings were assumed to behave as originally designed.
- Exceptional loads:
  - Exceptional loads, as defined by Table 3.1 of the CHBDC were not considered in this evaluation. This includes special vehicles, ice accretion and collision loading.

4.2 Evaluation Procedure

The evaluation was carried out as stipulated in CHBDC Section 14 - Evaluation with references to Section 3 – Loads, Section 8 – Concrete Structures, and Section 10 -Steel Structures. The condition of the bridge members was based on the site inspection (see Chapter 2). The structure was evaluated for a vehicle train (Evaluation Level 1), a two-unit vehicle (Evaluation Level 2), and a single-unit vehicle (Evaluation Level 3) as per Clause 14.9.1 of the CHBDC.

4.2.1 Assessing the Effects of Material Loss

The material properties of steel (i.e., elastic modulus, yield strength, etc.) are not influenced by the corrosion of the adjacent material[1]. However, corrosion does reduce the thickness of the steel member which changes the geometric properties that influence structural capacity (i.e. cross-sectional area, moment of inertia, radius of gyration, elastic, and plastic section moduli, etc.).

The subjective nature of assessing the extent of corrosion damage in steel bridges is not quantified with detailed guidelines in the governing bridge design standards (i.e., CHDBC and AASHTO). Consequently, designers must rely on technical literature and experience to develop rehabilitation strategies that satisfy the intent of the governing codes.

Prucz and Kulicki (1998)[2] present a method for accounting for the effects of corrosion loss in steel bridges. They propose a quantitative evaluation where the remaining capacity of a deteriorated detail or member may be obtained by multiplying the nominal as-built capacity by a local or member residual capacity factor (RCF). For tensile and compression members the RCF was the ratio of the remaining cross-sectional area to the original cross-sectional area. For bending members, the RCF is a ratio of remaining elastic section modulus to the original elastic section modulus.

CBCL adopted a similar approach for this analysis. An overall reduction to the member's gross area was applied to the main girders due to the uniform corrosion observed over a significant portion of their length (Figure 4-1). The remaining area, as measured from the corroded member, was used to statistically determine a representative minimum net area of the corroded cross-section. New geometric properties were determined for the remaining section (i.e. cross-sectional area, moment of inertia, radius of gyration, and elastic section modulus).

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[1] Melchers, R.E. 2003. Probabilistic Models for Corrosion in Structural Reliability Assessment-Part1: Empirical Models. Journal of Offshore Mechanics and Artic Engineering. ASME, 125: 265-271.

[2] Prucz, Z. & Kulicki, J. 1998. Accounting for Effects of Corrosion Section Loss in Steel Bridges. Transportation Research Record. Vol. 1624 (1): 101-109.

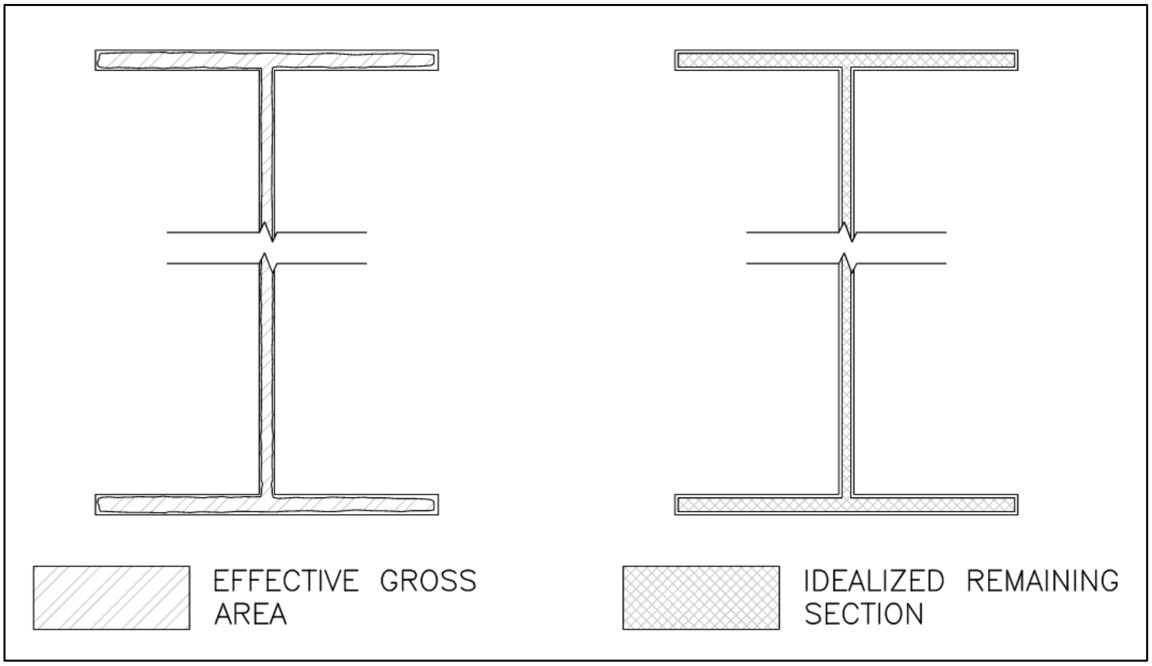


Figure 4-1: Effective Net Area for Assessing Strength of Corroded Members

4.2.2 Superstructure Evaluation

The superstructure of the bridge was analyzed using a 3-D finite element analysis (FEA) model of the structure. The results of the analysis were compared with manual code checks of the member resistances. The following tasks were completed for the superstructure evaluation:

- Creation of a 3-D FEA frame model of the structure with dead and live loading applied in accordance with CHBDC.
- Calculation of member section properties.
- Calculation of member resistances in accordance with the CHBDC.
- Determination of load factors for each member type based on their target reliability index.
- Determination of the utilization ratios for primary steel members at Ultimate Limit States (ULS 1) in accordance with the CHBDC.

4.2.2.1 Steel Material Properties

The existing drawings provided by the Department show the steel material specification for the main girder webs and flanges as ASTM A373 and the remaining steel components as CSA G40.4, which corresponds to a yield strength of 220 MPa and 230 MPa respectively as presented in the Historical Listing of Selected Structural Steels found in the CISC Handbook of Steel Construction. A clip from the hardcopy 'blueprint' of the original bridge drawing is provided in Figure 4-2. As discussed in Section 3.4, coupons were extracted from the East and West girders of the continuous span, and yield strength of 156 MPa was determined to be used for those members in the analysis.

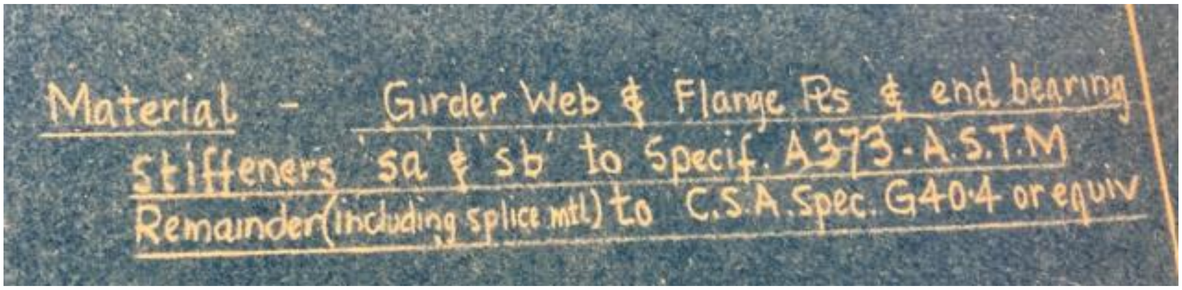


Figure 4-2: Original Drawing Material Specification

4.2.2.2 Steel Girder Member Evaluation Procedure

Main Girders – Continuous Span

The main girders on the Continuous Span run continuously over the pier supports. They were analyzed as non-composite flexural members in accordance with Clause 10.10 of the CHBDC as there are no details of shear connectors provided on the reference drawings. Vehicular and dead load is transferred to the girders as point loads at the transverse floor beam locations. The top flange of the girders was considered to be laterally supported at every second-floor beam, which also corresponds to the connection locations of the plan bracing. The bottom flange of the girders was considered laterally supported at every internal diaphragm location. CBCL has completed the evaluation based on the UT measurements collected by Tacten on two different occasions (Appendix F). The assumed section properties of the girders were determined at each panel point based on the nearest UT test location (Figure 4-3). The UT measurements taken on the latter date governed over the earlier measurements as it was suspected that the reporting of the location of the measurements was inaccurate.

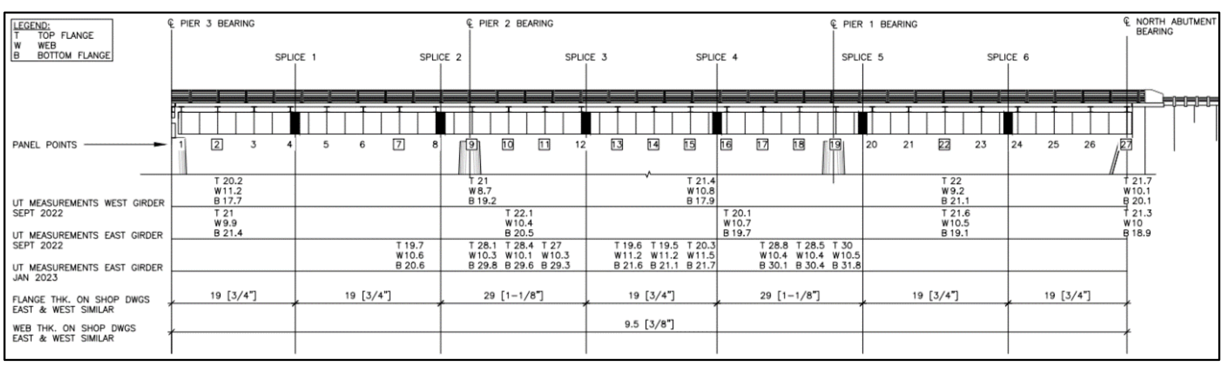


Figure 4-3: UT Readings used for Evaluation – Continuous Span

Main Girders – Swing Span

The main girders on the swing span run continuously over the pivot pier and are roller supported on each end at the south abutment and pier 3. They were analyzed as non-composite flexural members in accordance with Clause 10.10 of the CHBDC as there are no details of shear connectors provided on the reference drawings. The top flange and bottom flange of the girders were considered to be laterally supported at each floor beam and each internal diaphragm, respectively.

The UT readings on the swing span showed no appreciable corrosion loss on the top flanges, but the bottom flange on the west girder exhibited approximately 15% thickness loss. Based on this information, CBCL has completed the evaluation using the UT readings from two separate site visits (Appendix F). When no UT was available the section properties were developed based on the thicknesses shown on the shop drawings with an estimate of 10% thickness loss.

Floor Beams

The floor beams span transversely across the bridge deck. They were analyzed as non composite flexural members in accordance with Clause 10.10 of the CHBDC as there are no details of shear connectors provided on the reference drawings. The moment and shear demand on the floor beams were extracted from the FEA model. The cross section of the floor beams was reduced to account for corrosion loss based on the UT readings.

Stringers

The stringers are only present on the swing span. They were analyzed as non composite flexural members in accordance with Clause 10.10 of the CHBDC as there are no details of shear connectors provided on the reference drawings. The exterior stringers consist of channel sections located on the exterior of the main girders, and the interior stringers consist of W-sections that are located between the main girders. The stringers frame longitudinally with the deck between the floor beams. The moment and shear demand on the stringers were extracted from the FEA model. The cross section of the floor beams was reduced to account for corrosion loss based on the UT readings.

4.2.2.3 Concrete Deck Evaluation Procedure

The concrete deck evaluation was based on Clause 14.14.1.3 and appropriate clauses from Section 5 and 8 of the CHBDC. The reinforcing details and concrete strengths have been obtained from the reference drawings. The moment and shear demand on the deck was extracted from the FEA model.

4.2.3 Substructure Evaluation

The substructure evaluation involved an axial load analysis of the piers. The evaluation included the compressive capacity design and bearing checks with edge effects.

Axial Compressive Capacity

The original design drawings do not indicate any reinforcing steel in the pier and no reinforcing was witnessed in the pier shaft during the concrete scanning for the core extraction. Therefore, the pier design has assumed to be plain concrete. CHBDC provides no guidance on the design of plain concrete members therefore the evaluators consulted CAN/CSA A23.3-14 Design of Concrete Structures. The geometry of these bridge piers does not meet the aspect ratio requirements or height limitations applicable for use as 'Plain Concrete' in accordance with Clause 22 of the standard.

Nonetheless, a factored axial load resistance of the pier was computed in accordance with Clause 22.4.1.3 to gauge the sensitivity of the capacity of the pier. Due to the very severe erosion of the shafts the effective area has been reduced by 300 mm along each side and the triangular noses were omitted. The concrete design strength was inferred from the original design drawings as 21 MPa. The compressive strength from the material testing was not used in this analysis since the sample size required to obtain statistical reliability in accordance with CSA A23.1 would have been cost prohibitive and superfluous given the amount of reserve capacity obtained from an axial load analysis of a mass concrete pier.

Edge Bearing Capacity

The bearing capacity of the girders ends, supported at the edge of the pier 3 and at the abutments, has been evaluated in accordance with the methodology described in CPCI Design Manual[3]. The failure mechanism is illustrated in Figure 4-4. Edge reinforcing is present at the top of the pier/abutment; therefore, each girder is assumed to have a horizontal reaction (due to friction) at the point of bearing that is 20% of the vertical reaction.

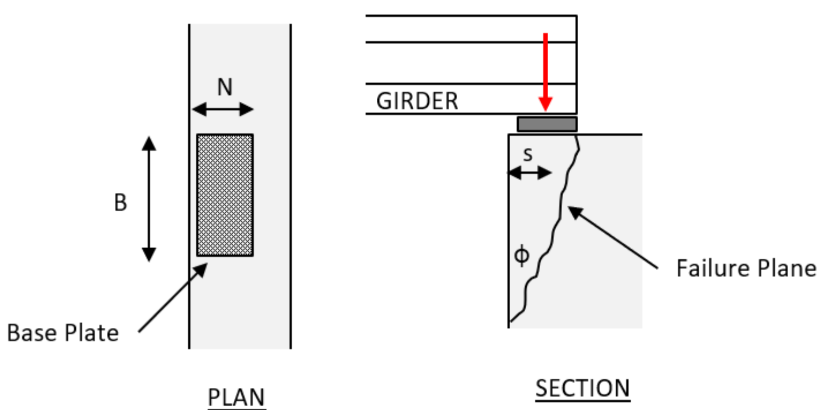


Figure 4-4: Edge Bearing Failure Plane

[3] Canadian Precast/Prestressed Concrete Institute (2017). CPCI Design Manual. 5th Edition. Ottawa, ON. Page 4-14.

4.2.4 Target Reliability Index

Section 14 of the CHBDC contains provisions for using a probabilistic framework to define load and resistance factors based on the bridge's historical performance, the condition of members, the mode of failure, and the importance of the member in the overall behaviour of the structure. This is accomplished using the target reliability index. In general, for a new structure, the target reliability is generally the same for all members. During a bridge evaluation, the Engineer can assign different values for each element based on certain criteria. The target reliability index is determined by assigning a value, 1 – 3 to three considerations:

System Behaviour - The effect of the element's failure on the entire structure

- Category S1, where element failure leads to total collapse. This includes failure of main members with no benefit from continuity or multiple-load paths.
- Category S2, where element failure probably will not lead to total collapse. This includes main load-carrying members in a multi-girder system or continuous main members in bending.
- Category S3, where element failure leads to local failure only. This includes deck slabs, stringers, and bearings in compression.

Element Behaviour - Consideration for the element's ductility

- Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning.
- Category E2, where the element being considered is subject to sudden failure with little or no warning but will retain post-failure capacity.
- Category E3, where the element being considered is subject to gradual failure with warning of probable failure. This can include steel beams in bending or shear, under-reinforced concrete in bending, decks, and steel in tension at gross section.

Inspection Level - Consideration of the level of inspection on the element

- Inspection Level INSP1, where a component is not inspectable. This can include hidden members not accessible for inspection, e.g., interior webs of adjacent box beams.
- Inspection Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator.
- Inspection Level INSP3, where the evaluator has directed the inspection of all critical and substandard components and final evaluation calculations account for all information obtained during this inspection.

A summary of assumptions and associated target reliability indices used for this evaluation is given in Table 4-1.

Table 4-1: Summary of Target Reliability Indices used in the Live Load Evaluation.

Members	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index
Main Girder – Continuous Span Main Girder – Swing Span	S1	E3	INSP3	3.00
Floor Beams Stringers Concrete Deck	S3	E3	INSP3	2.50

4.2.5 Load and Material Factors

4.2.5.1 Dead Load

The following dead loads were considered in the analysis:

- Self-weight of the steel members + 12% allowance for stiffeners, connections etc. as per C14.8.2.1 of the CHBDC commentary
- Reinforced concrete deck
- Concrete sidewalk and curbs
- Insulated waterline on west girder
- Conduit on east girder
- Aluminum barriers

Dead Load factors were applied as per the CHBDC's Table 14.7 and using the target reliability indices presented in Table 4-1. In instances where the dead load effect counteracts the effect due to transitory load the factors from the CHBDC's Table 3.3 were used as recommended by Clause 14.13.1 CHBDC.

4.2.5.2 Live Load

As specified in the RFP, Marystown River Bridge was to be evaluated using the CL1-625 design truck (Figure 4-5). The vehicle was assigned as a moving load in the analysis. As per Clause 14.9.4.1, two lanes are considered in accordance with the current intended use of the bridge. The CL1-625 truck load is applied to each of the two travel lanes in various positions. A multiple lane loading factor of 0.9 was applied to live loads in accordance with the CHBDC's Table 14.3 for Normal Traffic. Another load case that was analyzed used a reduced CL1-625 truck load (80%) applied as a moving load in combination with a uniform lane load which represented other vehicular traffic on the bridge. The bridge structure was also evaluated for a two-unit vehicle (Evaluation Level 2), and a single-unit vehicle (Evaluation Level 3) as per Clause 14.9.1 of the CHBDC.

To establish the highway class, the Department provided the average annual daily traffic (AADT) count of 6854. This volume classifies the highway as a Class A in accordance with clause 1.4.2.2 of the CHBDC. Class A lane loading was used in the evaluation but as discussed later in Section 4.4, this assumption has implications on the load posting of the bridge. As a caveat, and albeit a small sample size, this volume of traffic is much greater than what the bridge inspectors experienced over the five days while on site.

A Dynamic Load Allowance (DLA) of 1.25 was applied as per Section 3.8.4.5 of the CHBDC. The DLA was not applied to the CL1-625 in combination with lane load effects described above (as per CHBDC requirements). Live Load factors were determined in accordance with the CHBDC's Table 14.8, and the target reliability indices presented in Table 4-1 above and ranged from 1.35 to 1.49 in this evaluation.

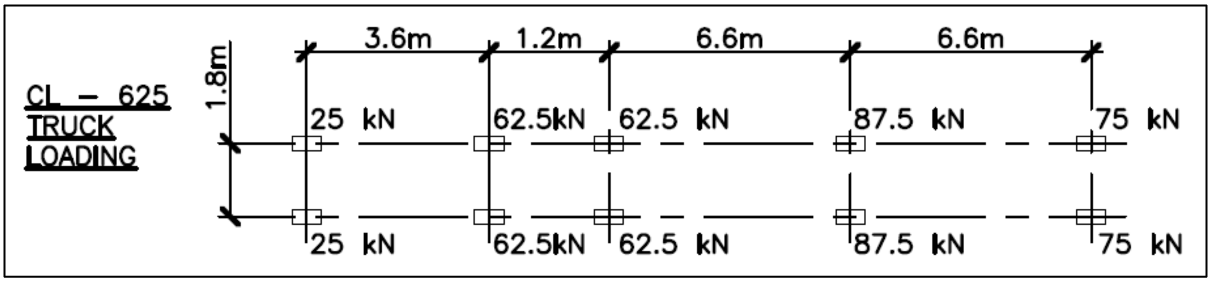


Figure 4-5: CL1-625 Truck Wheel Loading Diagram

4.2.6 Finite Element Analysis Model

The structural analysis for the live load evaluation was performed using a 3-D finite element analysis model of the full superstructure (Error! Reference source not found.) in the program LUSAS Bridge. The model was developed based on the existing drawings provided and field information retrieved during the site inspection. The supports were idealized as pinned supported for the fixed bearings and roller supported for the free bearings. The concrete deck and girder webs were modelled as shell elements. All other components such as girder flanges, floor beams, stringers and bracing were modelled as beam elements. Applicable live loads were modelled as a series of moving point loads in increments of 2 m.

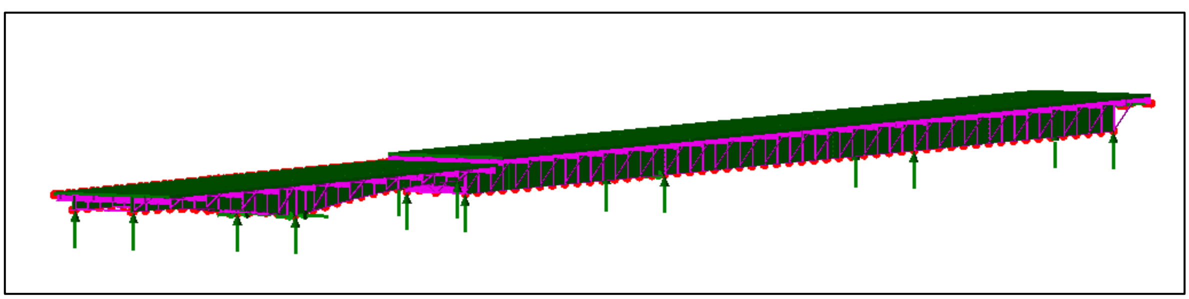


Figure 4-6: 3-D View of Marystown Harbour Bridge FEA Model in LUSAS Bridge.

4.2.7 Live Load Capacity Factors

Load factors were inputted into the FEA model as determined from Section 14 of the CHBDC. These load effects were extracted from the model and compared against factored member resistances that were calculated using the CHBDC.

Live Load Capacity Factors (LLCF) are determined per Section 14 of the CHBDC to evaluate the structure's capacity to carry additional live load (beyond the evaluated load). A LLCF above 1 suggests that the structure is capable of carrying additional live loads, while a LLCF below 1 indicates that the member or structure cannot theoretically safely support the specific live load being evaluated.

The LLCF is defined by the following formula:

$$F = \frac{\sum \mathrm{Rr} - \sum \alpha_{\mathrm{D}}\, \mathrm{D}}{\alpha_L\, L(1+I_D)}$$

Where:
- Rr = factored resistance of structural component
- $\alpha_D D$ = factored dead load
- $\alpha_L L$ = factored live load
- ID = dynamic component of live load

4.3 Evaluation Results

4.3.1 Superstructure Results Summary

The load evaluation of the bridge superstructure revealed the following results:

4.3.1.1 Continuous Span

- Both the east girder and the west girder were found to be overutilized in negative and positive bending, and combined shear and bending under evaluation levels 1, 2, and 3.
- Floor beams were found to have LLCF greater than 1 for both flexure and shear for evaluation level 1.
- A summary of the maximum utilization ratios for the continuous span girders can be found in Table 4-2.

Table 4-2: Continuous Span Girder Maximum Utilizations

Member	Evaluation Level	Location	Failure Mode	Utilization Ratio[1]
East Girder	1	Over Pier 1 & 2	Combined shear and Negative moment	1.58
East Girder	1	Over Pier 1 & 2	Negative moment	1.53
East Girder	1	Midspans of each span	Positive Bending	1.74
East Girder	2	Over Pier 1 & 2	Combined shear and Negative moment	1.53
East Girder	2	Over Pier 1 & 2	Negative moment	1.49
East Girder	2	Midspans of each span	Positive Bending	1.68
East Girder	3	Over Pier 1 & 2	Combined shear and Negative moment	1.41
East Girder	3	Over Pier 1 & 2	Negative moment	1.39
East Girder	3	Midspans of each span	Positive Bending	1.47

[1]Utilization ratios over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

Member	Evaluation Level	Location	Failure Mode	Utilization Ratio[1]
West Girder	1	Over Pier 1 & 2	Combined shear and Negative moment	1.42
West Girder	1	Over Pier 1 & 2	Negative moment	1.39
West Girder	1	Midspans of each span	Positive Bending	1.41
West Girder	2	Over Pier 1 & 2	Combined shear and Negative moment	1.38
West Girder	2	Over Pier 1 & 2	Negative moment	1.38
West Girder	2	Midspans of each span	Positive Bending	1.35
West Girder	3	Over Pier 1 & 2	Combined shear and Negative moment	1.22
West Girder	3	Over Pier 1 & 2	Negative moment	1.23
West Girder	3	Midspans of each span	Positive Bending	1.10

[1]Utilization ratios over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

The below diagram shows the governing utilizations based at various panel points along the East girder. Reference Figure 4-3 for panel point locations.

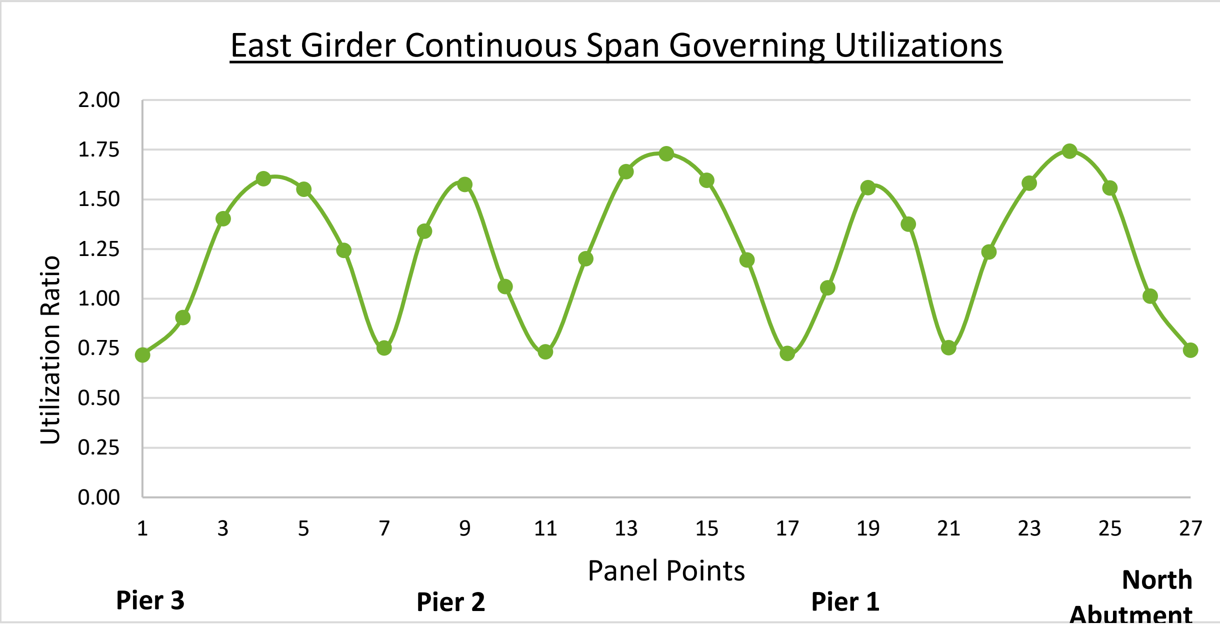


Figure 4-7: Panel Point Locations

4.3.1.2 Swing Span

- Both the east girder and the west girder was found to be overutilized for evaluation levels 1, 2, and 3 in positive bending and evaluation level 1 and 2 in shear. The bending failure occurs at mid-span between the south abutment and pivot pier and at mid-span between the pivot pier and pier 3. The shear failure occurs at the south abutment and pier 3.
- Both the west and east girder were found to have satisfactory capacity under negative bending for all evaluation levels.
- The exterior channel stringers were found to have a LLCF less than 1 for flexure under evaluation level 1.
- Floor beams and interior stringers were found to have LLCF greater than or equal to 1 for both flexure and shear under evaluation level 1.
- A summary of the maximum utilization ratios for the swing span girders can be found in
- **Table 4-3**.

Table 4-3: Swing Span Girder Maximum Utilizations

Member	Evaluation Level	Location	Failure Mode	Utilization Ratio[1]
East Girder	1	Mid-Span	Positive Bending	1.58
East Girder	1	South Abutment and Pier 3	Shear	1.13
East Girder	2	Mid-Span	Positive Bending	1.56
East Girder	2	South Abutment and Pier 3	Shear	1.07
East Girder	3	Mid-Span	Positive Bending	1.41
West Girder	1	Mid-Span	Positive Bending	1.50
West Girder	1	South Abutment and Pier 3	Shear	1.03
West Girder	2	Mid-Span	Positive Bending	1.49
West Girder	2	South Abutment and Pier 3	Shear	1.01
West Girder	3	Mid-Span	Positive Bending	1.34

[1]Utilization ratios over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

4.3.1.3 Concrete Deck

- The deck was found to have sufficient capacity for unrestricted traffic access.
- A summary of the maximum utilization ratios for the deck can be found in Table 4-4

Table 4-4: Deck Maximum Utilizations

Member	Evaluation Level	Failure Mode	Utilization Ratio[1]
Continuous Span Deck	1	Positive Bending	0.60
	1	Negative Bending	0.94
	1	Shear	0.98
Swing Span Deck	1	Positive Bending	0.47
	1	Negative Bending	0.84
	1	Shear	0.36

[1]Utilization ratios over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

4.3.2 Substructure Results Summary

4.3.2.1 Axial Capacity Load Evaluation

The piers do not meet the aspect ratio or height requirements to comply with the use of CSA A23.3 Clause 22 for plain concrete analysis. A23.3 permits a wall height to thickness ratio limit of 3 and a maximum wall height of 3 m. These bridge piers exceed these requirements by 100% and 144% respectively. The pier bridge was evaluated based on an envelope of reactions from the girders on the piers. The LLCF is provided in Table 4-5.

4.3.2.2 Pier Bearing Edge Load Evaluation

The edges of the piers at bearing locations were evaluated based on the maximum reaction from the FEM model for the Ultimate Limit State. The concrete bearing showed sufficient capacity at the edge of the piers. The LLCF is provided in Table 4-5.

Table 4-5: Summary Pier Concrete Bearing Check

ELEMENT	DESIGN CHECK	LLCF[2]
Piers 1, 2, & 3	Axial Capacity[1]	7.71
	Edge Bearing	1.70

[1] Does not comply with underlying geometric constraints of clause.
[2] LLCF factors under 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

4.4 Recommendation

Following the precious described analysis, CBCL determined the lowest Live Load Capacity Factor (LLCF) for the bridge. This is summarized below.

Live Load Capacity Factors – Class A Highway			
EVALUATION LEVEL	LEVEL 1	LEVEL 2	LEVEL 3
LLCF[1]	0.14	0.15	0.19

[1] LLCF factors under 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

As described in Section 4.2.7, a LLCF below 1 indicates that the member or structure cannot safely support the specific live load being evaluated. In such cases, a load posting is warranted to communicate the load that can safely cross the bridge.

However, when the LLCF for Evaluation Level 3 is less than 0.3, the Canadian Highway Bridge Design Code advises that consideration shall be given to closing the bridge. As such, CBCL recommends closing the bridge until such time that repairs have taken place to justify allowing traffic. Alternatively, based on the findings of this report, bridge replacement may be warranted.

In this case, the load evaluation findings and condition assessment show that considerable repairs would have to take place to allow for full highway loading. Repairing to this extent may also introduce a risk of damaging the structure's components that would otherwise be in fair condition. This is further explained in Chapter 8.

4.4.1.1 Emergency Vehicles

Following communication of this recommendation to the Department, CBCL were asked to review the possibility of emergency vehicles (i.e. fire trucks and ambulances) crossing the bridge in extenuating circumstances. This analysis is presented in a memo, along with the live load calculations, in Appendix M. The analysis reviewed specific vehicles weighed at a weigh scale. The analysis found that both vehicles may cross the bridge under the following circumstances:

- The bridge must be clear of any traffic when the vehicle is crossing.
- Only one vehicle can be on the bridge at a time and there is a monitor to enforce this during emergency situations.
- The bridge is barricaded/gated under non-emergency situations.
- The emergency vehicle must travel down the center of the bridge's travel way.
- The vehicle must travel at a speed less than 20 km/hr.

CBCL recommends completing bi-annual inspections to maintain intended pedestrian and emergency vehicle usage.

Chapter 5 Fatigue Evaluation

The fatigue evaluation of existing steel bridges is stipulated in Section 14.18 of the CHBDC. This section states that a bridge is to be assessed for fatigue and remaining fatigue life at the fatigue limit state using the appropriate method where there are fatigue-prone details or physical evidence of fatigue-related defects.

5.1 Fatigue Analysis Procedure

The fatigue analysis was conducted in accordance with Section 14.18 with supporting clauses from Section 10.17 of the CHBDC. For fatigue limit state the traffic load was calculated using an elastic analysis in the FEA model for one CL1-625 truck increased by the dynamic load allowance and placed at the center of one travelled lane. The detail categories used in this analysis to check load induced fatigue are shown in Table 5-1.

Table 5-1: Detail categories for load induced fatigue analysis.

General Condition	Situation	Detail Category
Plain member	At re-entrant corners of copes with a radius ≥ 35 mm and ground smooth	E1
Fillet-welded connections with welds normal and/or parallel to the direction of stress	At the toe of transverse stiffener to flange and transverse stiffener to web welds	C1
Plain Member	Base metal of gross section with rolled or cleaned surfaces.	A
Longitudinally loaded fillet welded attachments	Base metal at details attached by fillet welds. When the detail length in the direction of applied stress is greater than either 12 times the detail thickness or 100 mm. Detail thickness < 25 mm.	E
Mechanically fastened connections	Holes drilled or punched and reamed to size	B

The Department provided an average annual daily traffic (AADT) for this structure as 6,854. The fatigue analysis used the following parameters:

- Highway Class A with Average Daily Truck Traffic (ADTT) = 4000 (Table 10.6 of the CHBDC); this was used as the AADT provided by the Department satisfies Clause 1.4.2.2 for Highway Class A.
- Years of applied ADTT for the calculation of design stress cycles = 75 years; This was conservatively used to assess the potential for future fatigue issues within the remaining service life of the structure. The actual fatigue evaluation would be based on the structure's current age (65 years).

5.2 Fatigue Analysis Results Summary

Several members have calculated fatigue stress ranges ($0.52C_L f_{sr}$) greater than their fatigue stress range resistance (F_{sr}) (Table 5-2). At the time of the inspection no cracks were observed through field observations and magnetic particle testing completed. Detailed fatigue calculations are provided in Appendix N.

Table 5-2: Summary of Fatigue Analysis Results

Span	Member	Detail Category	$0.52C_L f_{sr} < F_{sr}$?
Swing	Girder Top Flange	C1	NO
Swing	Girder Bottom Flange	C1	NO
Swing	Interior Portion of Floor Beam	E1	NO
Swing	Cantilever Portion of Floor Beam	C1	YES
Swing	Interior Stringers	A	YES
Swing	Exterior Stringers	A	YES
Continuous	Top Flange	E	NO
Continuous	Bottom Flange	C1	NO
Continuous	Interior Portion of Floor Beam	E	NO
Continuous	Cantilever Portion of Floor Beam	C1	NO

As per Clause 10.17.2.5 of the CHDBC, the width to thickness ratios of transversely stiffened webs, h/w, was checked. Fatigue testing on beams with unstiffened webs has demonstrated that a slenderness ratio greater than $3150/\sqrt{Fy}$ are susceptible to fatigue cracking at the flange to web junction. Since, the swing span web is tapered this check was completed at the location where the web height is greatest (at the pivot pier). Any location where the height of the swing span exceeds the code prescribed limit may be susceptible to fatigue cracking at the flange to web junction. These results are summarized in Table 5-3.

Table 5-3: Summary of Fatigue Analysis Results

Span	h (mm)	w (mm)	h/w	$3150/\sqrt{Fy}$	h/w < $3150/\sqrt{Fy}$?
Continuous	1790.7	9.53	188	212	YES
Swing	2044.7	9.53	215	212	NO

Chapter 6 Seismic Analysis

6.1 Scope of Work

The primary objective of the seismic analysis was to complete a preliminary assessment of the seismic risk of Marystown Harbour Bridge. Given the age of the bridge and the low seismicity of the region, it is unlikely that the original design gave any attention to seismic effects. Therefore, a structural analysis of the bridge was carried out to determine how the structure would behave during a seismic event and to identify any critical members that could experience yielding and/or failure. This study will provide the Department with information to identify the need for further evaluation regarding seismic safety and the justification for a seismic strengthening strategy. The scope of work involved a desktop seismic load evaluation in accordance with Section 4 of the CHBDC.

6.2 Analysis Procedure

Marystown Harbour Bridge is specified in the RFP as a *major route bridge* and is defined as seismically *regular* in accordance with CHBDC Table 4.14. The modal analysis of the bridge defined a period of 0.69s in the transverse direction which placed the bridge in Seismic Performance Category 2 (SPC2).

The seismic evaluation was performed as an elastic dynamic analysis using the multi-mode elastic response spectral analysis approach. A 3-D finite element analysis model was used to represent the structure. The continuous span and the swing span are independent structures and were therefore modelled separately (
Figure 6-1). The load effects on pier 3 were superimposed from each model. The superstructure was modeled based on the geometry and steel sizes from the original drawings and field findings. The abutments and piers contribute to the seismic stiffness of the bridge and were modelled based on the geometry of the original drawings. Stiffness modifiers were used to account for the deterioration in the structures. The superstructure was connected to the piers and abutments by assigning link elements that simulate the associated bearings that exist on the 'real' structure.

The model was subjected to an acceleration response design spectrum based on a site class C[4] and the spectral acceleration coefficients obtained for the site[5]. The seismic hazard level was assigned as 10% in 50-year probability of exceedance in accordance with clause 4.11.3 for SPC2 bridges. The seismic design checks were carried out using the force-based design approach defined in CHDBC.

The response modification factor, R, for a force-based design depends on the ability of the substructure element to develop an appropriate level of ductility. For this analysis R was conservatively chosen as the minimum value of 2.0 a specified in Cl 4.4.7.2. The importance factor, I_E, was taken as 1.5 for major route bridges. The mass of the bridge was determined from the dead load of the steel girders, bracing, concrete deck, piers, and abutments. The soil-structure interaction of the bridge was idealized as fixed supports for pier foundations doweled into bedrock. The abutments and pier 1 are supported on piles. The depth of fixity of the piles was taken as twelve times the width of the pile. No field investigation of the dynamic characteristics of the soil was conducted or considered in this analysis.

This seismic evaluation was based on the following load combination:

- ULS5: 1.20D + 1.0EQ

6.2.1.1 Seismic Force resisting Element Evaluation Procedure

The critical seismic force resisting elements that were evaluated in this analysis include:

- Bearing anchorage: four 25 mm diameter A307 anchor rods.
- Diaphragm beam at each substructure element
- Piers

Bearing Anchorage

Each main girder on the Continuous Span is secured to a substructure element (i.e., abutment or pier) with four 25mm diameter anchor rods. The anchor grade is not provided on the drawings therefore it was assumed to be grade ASTM A307 with an ultimate tensile strength of 400 MPa. The embedment depth of the anchors is not specified on the original documents and therefore tensile pull-out, or concrete breakout cannot be determined. The only check that can be performed with a sense of reliability is the shear capacity of the anchor rod. The bearings at the swing span were inspected during the site assessment but the anchorage mechanism was determined inconclusive. All the hardware from the original operable swing stage is still in place but it is unclear how the rotating mechanism has been disabled. Therefore, this assessment will be limited to the continuous span.

[4] Site Class C as specified by NLDTI Bridge Office
[5] Spectral acceleration values obtained from https://earthquakescanada.nrcan.gc.ca

Diaphragm Beam

The main girders are braced at each abutment and pier with a steel wide flange diaphragm beam. This beam transfers lateral load to each bearing and hence performs as an axial loaded member in compression.

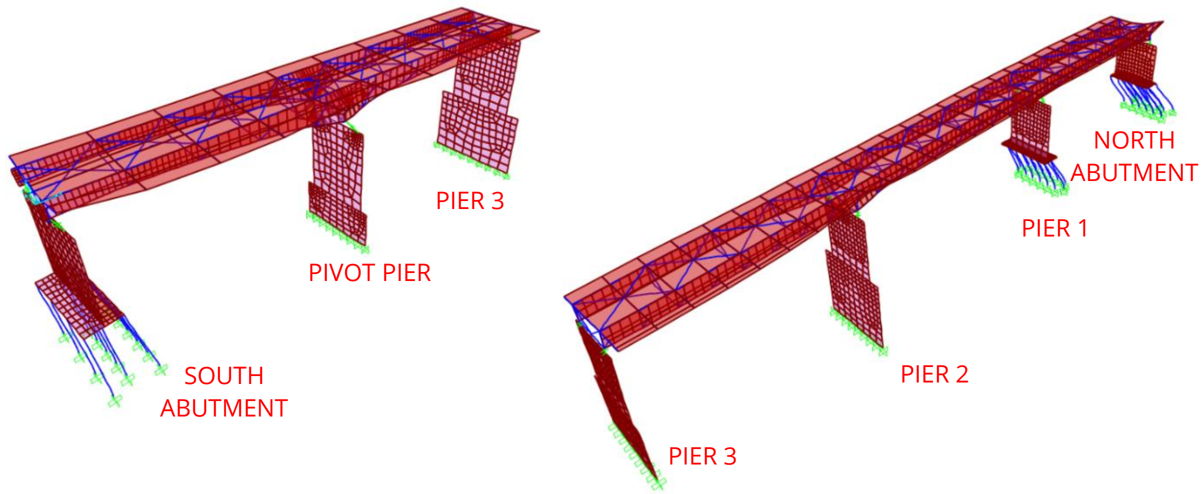
Piers

The piers are cantilevered structures that transfer lateral forces on the superstructure to the foundation. The piers must resist overturning and diagonal shear forces. Stability due to overturning at each substructure element is checked with load combination ULS5. For the elements with piles (South abutment, North abutment, and Pier 1), no uplift resistance was conservatively assumed for the piles. The piers are unreinforced concrete members and cannot be evaluated for strength since Clause 22.1.1 from CSA A23.3 prohibits the use of plain concrete in seismic resisting members.

6.3 Seismic Evaluation Results

The period in the transverse direction of the continuous span and the swing span was found to be 0.68 seconds and 0.63 seconds respectively with a seismic base shear of 503 kN and 466 kN respectively.

A summary of the highest utilizations for each seismic force resisting element is present in
Table **6-1**. From the results, the steel diaphragm beam and steel anchor rods at pier 1, 2, and 3 appeared to perform satisfactory at each substructure element. The piers are also not vulnerable to overturning during a seismic event. It can be concluded that this structure meets the criteria (within the limitations of the scope of work) for SPC2. Detailed calculations of the seismic analysis are provided in Appendix O.



a) Swing Span Model b) Continuous Span Model

Figure 6-1: FEA Seismic Model - Mode Shape 1

Table 6-1: Summary of Main Element Max Seismic Utilizations

Element	Location	Failure Mode	Utilization[1]
Bearing Anchorage	Pier 2	Shear	0.39
Diaphragm Beam	Pier 2	Compression	0.11
Pier	Pier 3	Overturning	0.19

[1]Utilization ratios over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

Chapter 7 Splice Plate Analysis

7.1 Scope of Work

A splice plate analysis was completed for each unique splice plate joining the main girders on both the continuous and swing span as per Clause 10.18 of the CHBDC. The splice plate analysis focused on the main girders and did not include an analysis of the gussets connecting any of the secondary (i.e., plan bracing) members. The scope of work also did not include any UT measurements on the splice plates or bolts.

7.2 Analysis Procedure

The following tasks were completed as part of the splice plate evaluation:

- Load results have been extracted from the FEA model at each splice location.
- Live load factors have been determined for each splice based on its target reliability index. The splice plate analysis has used the same target reliability indices as the main girders.
- Splice plate resistances have been calculated in accordance with Clause 10.18 of the CHBDC.

The following assumptions were made to complete the analysis:

- When UT readings are not available, splice plates assumed to have a thickness loss of 20% to account for corrosion.
- Bolt sizes were reduced to 19 mm diameter to account for corrosion.
- Splice plate dimensions were taken from the existing drawings provided by the Department.
- The splice plates on the top and bottom flanges are not rectangular and the bolt patterns are irregular. To evaluate these splice plates CBCL used smaller rectangular plates. The smaller plates have equal or smaller cross sections and shear area making this approach conservative. The geometry and bolt pattern used in the analysis is shown in Appendix P.

The following resistance checks were performed for the splice plate analysis:

- Tensile resistance of girder flanges (ULS)
    - a) Fracture of the net section
    - b) Block shear
- Tensile resistance of splice plates (ULS)

- a) Yielding of the gross section
- b) Fracture of the net section
- c) Block Shear

- Splice Plate Fatigue (FLS)
- Flange Bolts
  - a) Bolts in shear and bearing (ULS)

7.3 Splice Nomenclature

Splices on the east and west girders for the continuous and swing span are identified as shown in Figure 7-1: Existing Splice Plate Locations

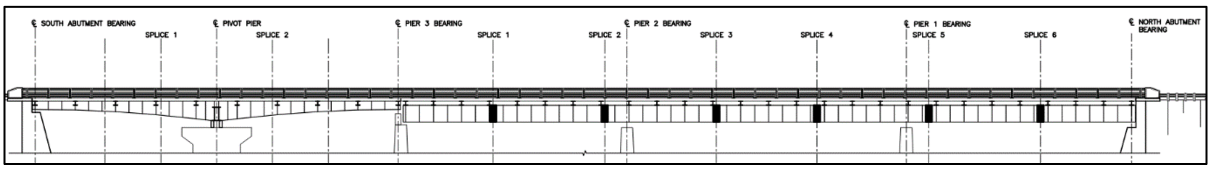


Figure 7-1: Existing Splice Plate Locations

7.4 Splice Material Properties

The original drawings show the splice plate material as G40.4. which corresponds to a yield strength of 230 MPa. The holes in the splice plates are specified as 15/16" in diameter.

The original drawings show the splice bolts as 7/8" diameter high strength bolts. The bolts are not specified as A325 bolts on the original drawings, but the A325 bolt markings were visible during the inspection and were therefore used for the analysis.

After the completion of the preliminary evaluation, CBCL returned to site in January 2023 to complete steel coupon testing on the East and West continuous girders. A yield strength of 156 MPa was determined. Although coupons were not extracted from the splice plates, CBCL used the yield strength of 156 MPa for the splice plate analysis due to the discrepancy between the steel coupon results and the original drawings.

7.5 Summary of Splice Plate Analysis Results

The maximum utilization ratios both the continuous and swing span are shown in Table 7-1.
Table 7-1: Splice Plate Max Utilizations

Swing Span		
Splice ID	**Max. Utilization Ratio[1]**	**Governing Failure Mode**
Splice 1 & 2 – East Girder	1.32	Yielding of Gross Section
Splice 1 & 2 – West Girder	1.00	Yielding of Gross Section
Continuous Span		
Splice ID	**Max. Utilization Ratio[1]**	**Governing Failure Mode**
Splice 1 – East Girder	1.86	Yielding of Gross Section
Splice 2 – East Girder	1.01	Yielding of Gross Section
Splice 5 – East Girder	1.10	Yielding of Gross Section
Splice 6 – East Girder	1.91	Yielding of Gross Section
Splice 1 – West Girder	1.45	Yielding of Gross Section
Splice 2 – West Girder	1.16	Yielding of Gross Section
Splice 3 – West Girder	1.26	Fatigue
Splice 4 – West Girder	1.13	Fatigue
Splice 5 – West Girder	1.14	Yielding of Gross Section
Splice 6 – West Girder	1.52	Yielding of Gross Section

[1]Utilization factors over 1 indicate that the member discussed does not have the theoretical capacity to support the evaluation loads.

Chapter 8 Rehabilitation

The contents of this chapter are based on an early draft of this report, prior to steel testing and recommendation for bridge closure. Nonetheless, the rehabilitation strategy is appropriate if the Department decides to extend the life of the bridge.

This chapter presents a rehabilitation strategy to enhance the long-term durability and performance of the bridge to achieve a service life until the year 2050 as specified in the RFP. The rehabilitation strategy is based on the results of the visual inspection, materials testing, and load evaluation. The primary elements of the rehabilitation strategy include:

- Traffic barrier replacement
- Bearing Replacement
- Abutment and deck concrete repairs
- Bridge approach improvements
- Girder strengthening
- Secondary steel replacement
- Recoating program
- Pier encapsulation

The proposed rehabilitation is illustrated in a concept drawing in Appendix Q. As described in the following sections, considerable rehabilitation work is required to allow for full highway loading. Repairing to this extent may also introduce a risk of damaging the structure's components that would otherwise be in fair condition.

8.1 Traffic Barrier Replacement

The existing four rail aluminum barrier system does not meet TL-4 crash test criteria of CHBDC. CBCL proposes to replace the existing system with the Departments standard four rail galvanized HSS barrier system. The posts are proposed to be anchored into the existing deck using adhesive anchors sized for the code specified anchorage force requirements.

8.2 Bearing Replacement

The condition assessment identified that all bearings were severely corroded and potentially seized at the North abutment and not free to move at the south abutment or at pier 3. The deterioration is beyond repair and replacement is the only option to restoring the structure's ability to translate effectively. It is estimated that all 12 bearings will need to be replaced.

8.3 Abutment and Deck Concrete Repair

The concrete material testing showed that the abutment concrete is of poor quality and not suitable for the environment in which it is in service. The entrained air void network has been measured to be substandard, meaning that the concrete members are susceptible to freeze/thaw damage and the chloride ion concentrations are high enough to promote corrosion in the reinforcing steel. Therefore, we do not recommend performing localized repairs to the abutments because freeze-thaw damage and reinforcing corrosion is already significant. The presence of ASR was confirmed but the advanced testing was not completed as part of this scope to determine if the reactivity was still active. In the case that ASR is no longer active a possible repair solution could be to encapsulate the abutments with a layer of new concrete with shrinkage/temperature reinforcing by first removing all loose and spalled concrete. The purpose of this new layer is to protect the substructure from further chloride ingress and freezing and thawing damage. As chloride ion concentrations have been measured to be of concern, it is recommended that the installation of galvanic anodes into the repair be used.

The deck topping was in relatively poor to fair condition with localized areas of deterioration. It is recommended that all defects noted as “poor” in the inspection report be repaired to prevent further deterioration of the concrete. The recommended procedure to rehabilitate localized deteriorated concrete will generally be as follows:

1. Department representative to delineate areas for repair;
2. Saw cut the perimeter of the repair area to a minimum on 25 mm depth;
3. Chip out concrete with pneumatic or electric chisel;
4. If exposed reinforcing bars are corroded the concrete surrounding the bars should be fully removed to provide a clearance of 25 mm;
5. Replace corroded reinforcing as directed by engineer;
6. Remove loose and bond inhibiting material from the bars and concrete surface; and
7. Mix, apply, and cure repair grout as per manufacturers written instructions.

The concrete test results revealed high chloride ion content in the slab which predicts that more delamination and spalling will occur in the future. In lieu, of scarifying and replacing the full bridge deck topping at this time, CBCL proposes to repair the ‘poor’ areas present at this time and conduct additional repairs in the future when more areas become deteriorated.

8.4 Approaches

CBCL recommends that smoother transitions be installed on the approaches of the bridge. This will improve the rideability of the bridge and decrease dynamic loading effects. The existing asphalt should be removed and replaced with a traditional 300 mm thick reinforced concrete approach slab and new asphalt. The approach slab will prevent settlement at the ballast wall and ensure an even transition. The ballast wall may need to be extended slightly to match the new height of the bridge deck depending on which deck replacement option is chosen. An expansion joint will need to be installed to connect the bridge to the approach.

8.5 Girder Strengthening

The rehabilitation strategy for these girders is to sandblast and reinforce the full length of the flanges and replace the splice plates. There is a significant amount of rust stratification and formation on the girder flanges that would make manual removal with grinders to a 'near white' condition suitable for welding a very onerous and impractical task. CBCL recommends sandblasting to remove the deleterious steel material. Reinforcing would then involve field welding/bolting new steel plates to the underside of the existing bottom chord and to the underside of the top flange (each side of web). Plate sizes would be determined during detailed design and will be sized to restore/enhance the structural capacity to the girders. This would also have to be designed to consider the fatigue life deficiencies identified in this report. The splice plates and bolts would need to be replaced. Each girder will have to be temporarily supported on brackets for the splices connections to be replaced. A significant amount of rehabilitation work is required to restore/enhance the structural capacity of the girders. Repairing the girders will require a significant amount of welding which may introduce a risk of damaging components of the structure that would otherwise be in fair condition.

8.6 Replacement of Secondary Steel

The condition assessment identified that all the diaphragm members were severely corroded. The deterioration is beyond repair and replacement is the only option to restoring the structure's stability under negative moment, lateral wind load, and seismic forces. It is estimated that 27 diaphragms need to be replaced in the continuous span and 10 diaphragms in the swing span. The horizontal cross-bracing was also found to be severely corroded and recommended to be replaced. The replaced is assumed to be 'like for like'.

8.7 Re-Coating

Due to this widespread poor condition, localized touch-up or a maintenance recoating will not suffice. The only option to restore the coating is to completely recoat the steel members of the bridge structure. The bridge coating is important, not only from an aesthetic perspective, but to seal the surface of the ferrous metal components from the rust that can form due to atmospheric moisture and to delay the onset of corrosion.

Recoating will require a sand blast surface preparation (SSPC-SP10)[6] to remove the remaining paint. The removed coating, mill scale, and corrosion will need to be separated from the abrasive and stored for disposal. The hazardous materials test on the existing paint samples confirmed the presence of lead in concentrations above the federal and provincial guidelines and therefore the removed paint will have to be treated and disposed of as hazardous waste. Measures must be taken, such as a hoarding structure, to ensure that the removed coating will not containment the environment below.

CBCL recommends a three-coat paint system that is qualified by The Northeast Protective Coating Committee (NEPCOAT)[7] and ISO-12944 – "Corrosion Protection of Steel By Protective Paint"[8]:

- Zinc primer (75 μm DFT, Organic-zinc primer)
- Epoxy mid-coat (200 μm DFT)
- Urethane topcoat (60 μm DFT, isocyanate fee with high aesthetic durability)

All areas to be painted will be hoarded to protect the environment from paint removal and coating activities. Containment requirements will follow SSPC Guide No.6 – *Guide for Containing Surface Preparation Debris Generated during Paint Removal Operations* and meet provincial regulations. The hoarding will be part of the access scaffold system and required to be heated when exterior temperatures fall below the manufacturers recommend curing temperatures for the paint products. Figure 8-1 shows a typical hoarding system for a truss recoating project completed in New Brunswick which is like the hoarding that may be required for Marystown Harbour Bridge. An anticipated coating program is presented in Table 8-1 to ensure that the coating functions properly over the service life of the bridge.

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[6] Surface preparation designation as specified by The Association for Materials Protection and Performance (AMPP), formerly, The Society for Protective Coatings (SSPC)

[7] NEPCOAT is an affiliation of northeastern states in the USA, for the purpose of developing acceptance/testing criteria of protective coating for use on highway bridge steel

[8] ISO 12944 standard addresses protective paint systems that can prevent corrosion in carbon and low-alloy structural steel

Table 8-1: Anticipated 30-year Re-Coating Program

Coating Activity	Year	Description
Original	0	Original shop applied coating system
Spot Touch Up	15	Spot coat sections of rust
Maintenance Recoat	20	Spot prime and full re-coat

Figure 8-1: Typical Hoarding System on Mirimichi Bridge, NB (circa 2013)

8.8 Pier Rehabilitation

The bridge inspection and material testing program concluded that the bridge pier concrete is in poor to very poor condition. To rehabilitate the pier, it is proposed to encapsulate the existing concrete in a shell of new reinforced concrete specifically designed for a marine environment. Encapsulating the existing concrete affords greater structural capacity due to confinement of the piers and enhanced durability due to protection from environmental forces. CBCL recommends that all loose and spalled concrete should be removed from the shaft walls and encapsulated in a layer of new concrete with GFRP shrinkage/temperature reinforcing. This purpose of this new layer is to protect the substructure from further chloride ingress, freezing and thawing effects, and ice abrasion.

It was noted that ASR was found to be present in the pier cores. Therefore, for this option to be viable, more extensive ASR testing is required to determine if the expansive agents have stopped expanding. The timeline for ASR testing is approximately 1 year. If ASR is still active, then the only recourse is to replace the piers.

8.8.1.1 SWOT Analysis – Pier Rehabilitation

Strengths

- Increase in substructure structural capacity
- New concrete will be designed to be durable for the environment and require low maintenance
- Minor disruption to traffic
- Lower carbon footprint by re-using concrete superstructure

Weaknesses

- Delays in construction schedule based on severity of the current/sea state
- Increased dead load may require additional piles on pier P1. Further geotechnical investigation required

Opportunities

- Develop a technique for rehabilitating piers in a marine environment that could be published in peer-reviewed literature or presented at a conference to highlight the Departments technical portfolio.
- Work with Memorial University Faculty of Engineering to support their state-of-the-art research program in ice interaction with concrete structures
- Explore the use of shielding material such as ultra high-performance concrete of HDPE panels to reduce the effect of ice/wave abrasion.

Threats

- Additional ASR testing is required to determine if all expansion has stopped
- Weather delays may threaten the project schedule
- Strong tides through the channel could impact the construction of underwater formwork
- Repairing the piers may result in large removal areas of unsound concrete, comprising stability during construction

Chapter 9 Replacement

This chapter presents a feasibility study for replacing Marystown Harbour Bridge with a new structure. The feasibility study assesses the best materials, optimized span lengths, and substructure types. Three (3) options have been evaluated for the Marystown Harbour bridge replacement:

- **Concept 1:** *Three Span Steel Box Girder*
- **Concept 2:** *Four Span Prestressed Concrete Girder Bridge*
- **Concept 3:** *Five Span Prestressed Concrete Girder Bridge*

The merits of integral and semi-integral abutments for each structure are discussed and recommendations are provided. Consideration is also provided for the pier type for the chosen superstructure concept.

The assumed design criteria for the bridge replacement includes the following:

- Design as per CHBDC for a design life of 75 years;
- Similar total bridge length as the existing structure as specified in the RFP;
- Geotechnical design is preliminary and based on the existing bridge construction;
- Assumed existing bridge hydraulic opening meets CHBDC and the Departments criteria for freeboard;
- Concrete as per the NLDTI standard specification;
- No expansion joints on the bridge deck (i.e., integral, or semi-integral abutments);
- Galvanized reinforcing in deck slab and abutments and GFRP in piers;
- NLDTI Standard Steel Barrier positioned on the bridge deck to provide an overhang and drip edge at the wingwalls;
- Asphalt thickness of 110 mm; and
- Bridge deck width to match existing (Figure 9-1)

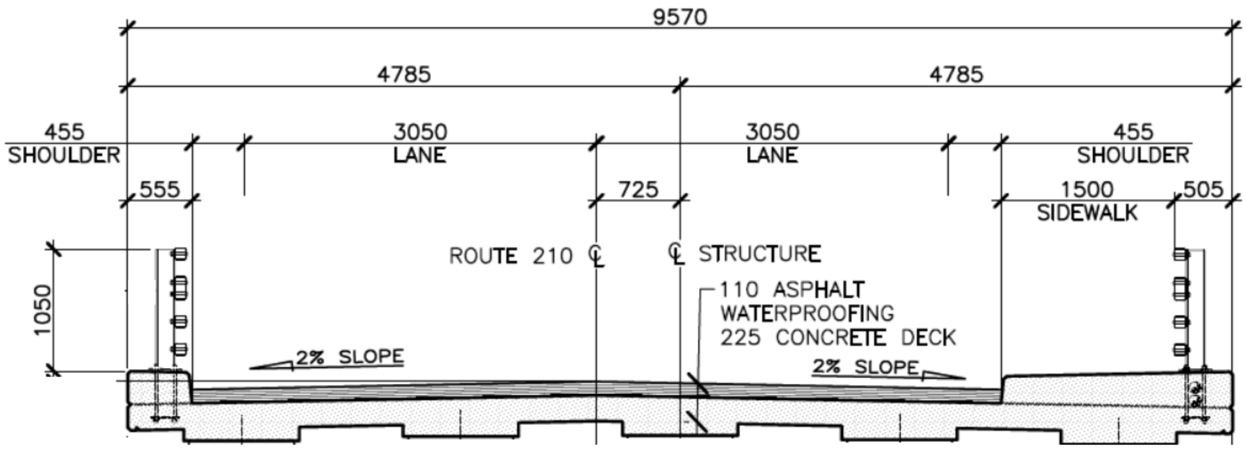


Figure 9-1: Bridge Deck Profile

9.1 Replacement Concepts

9.1.1 Replacement Concept 1 – Two Span Steel Box Girder

Concept 1 proposes the use of a two-span continuous structure, constructed from two trapezoidal steel box girders (60 m each) that act compositely with a 225 mm thick reinforced concrete deck. The girders will be supported on reinforced concrete abutments at each end and one intermediate pier. Based on preliminary calculations, the girder will be approximately 2100 mm deep bringing the total bridge height to 2435 mm which is approximately the same structural depth as the existing bridge and therefore, no adjustment will be required to the vertical road alignment.

At the time of this report there are no steel fabrication facilities in NL that have obtained the CSA S6-19 mandated quality management system certification issued by the Canadian Institute of Steel Construction in the category of steel bridges. The closest steel fabricator with the necessary credentials for fabrication of steel bridge girders is Cherubini Metal Works in Halifax, NS and Modular Fabrication in Miramichi, NB. Transporting girders of this size is very challenging due to the requirement of special permits, negotiations with shipping companies (i.e. Marine Atlantic, Oceanex, etc), and the difficulty of manoeuvring such large and heavy objects. Historically, box girders have been shipped to the province via Marine Atlantic's ferry service and transported on a flat bed to the bridge site. Due to their length, these girders will likely be shipped in segments (two end sections and one middle section) and spliced on site prior to erection. It is expected that the girders will be erected by launching from an abutment or 'leap frogging' from the existing bridge. Bridge contractors in the province have experience with steel box girder bridge construction with numerous successful bridge replacements in NL being constructed in the last decade (Table 9-1).

Table 9-1: List of Steel Box Girders Constructed in NL since 2010

Bridge	Span (m)	Year Built	Approx Location
ES Spencer Bridge	75	2013	Glovertown Exit (TCH)
Northwest River Bridge	61	2016	Terra Nova National Park (TCH)
Deer Arm Brook Bridge	62	2017	Gros Morne National Park (Route 430)
Sir Robert Bond Bridge	210 (3 Span)	2017	Bishops Falls (TCH)
Sandy Lake Narrows Bridge	75	2017	Howley (Route 401)
Bakers Brook Bridge	44	2018	Gros Morne National Park (Route 430)
Rocky Barachois Bridge	42	2019	Gros Morne National Park (Route 430)
Dicks Brook Bridge	68	2020	Gros Morne National Park (Route 430)
Shoal Harbour River Bridge (under construction)	65	Planned 2023	Clarenville (TCH)
Western Brook Bridge	54	Planned 2024	Gros Morne National Park (Route 430)

9.1.1.1 Protective Coating

To protect the steel box girder material against atmospheric corrosion, the Department specifies a shop applied three-coat paint system that is qualified by The Northeast Protective Coating Committee (NEPCOAT)[9] and by ISO-12944 – "Corrosion Protection of Steel By Protective Paint"[10]. This system includes the following layers:

- Inorganic zinc primer;
- Epoxy mid-coat; and

[9] NEPCOAT is an affiliation of northeastern states in the USA, for the purpose of developing acceptance/testing criteria of protective coating for use on highway bridge steel

[10] ISO 12944 standard addresses protective paint systems that can prevent corrosion in carbon and low-alloy structural steel

- Polyurethane topcoat,

According to ISO-12944 the environmental exposure condition of Marystown Harbour Bridge would be classified as Type 5-M: very high (coastal and offshore areas with high salinity). The bridge spans across brackish water and will be subjected to sea spray on the structure. According to data published by AMPP[11] the proposed coating system has a practical service life of approximately 15 years in this environment. The practical service life is defined as the time it takes for 5 to 10% of the coating to breakdown and active rusting of the substrate to become present. Spot touch-ups are generally considered to occur at the practical service life. A maintenance recoat, whereby rust spots are primed, and the steel is re-coated, is estimated to occur at 133% of the practical life. A full recoat is generally required at 50% of the practical life past the maintenance recoat. For a full recoat, the existing coating is completely removed down to bare steel and replaced. A summary of the coating program anticipated for the 75-year design life of this bridge is provided in Table 9-2.

Table 9-2: Anticipated 75-year Re-Coating Program

Coating Activity	Year	Description
Original	0	Original shop applied coating system
Spot Touch Up	15	Spot coat sections of rust
Maintenance Recoat	20	Spot prime and full re-coat
Full Re-coat	27	Total coating removal and replacement
Spot Touch Up	42	Spot coat sections of rust
Maintenance Re-coat	47	Spot prime and full re-coat
Full Re-coat	54	Total coating removal and replacement
Spot Touch Up	70	Spot coat sections of rust

[11] Helsel et al. (2014). "Expected Service Life and Cost Considerations for Maintenance and New Construction Protective Coating Work". Paper No. 4088. Corrosion Conference. San Antonio Texas. March 2014.

9.1.1.2 SWOT Analysis – Superstructure Concept 1

Characteristics of this concept have been assembled in the following SWOT analysis:

<u>Strengths – Concept 1</u>

Maintenance, Repairs, and Inspection Features:

- Affords longer spans which reduce the amount of substructure elements to inspect and maintain
- Two span structure has less piers than the other options, increases the hydraulic opening, and mitigates any damage or flood issues associated with ice build-up.
- Steel box girders provide inspection hatches to access the interior of the girder which improve inspection capabilities
- Box girders offer higher corrosion resistance than other steel structures because half of the steel surface is not exposed to airborne chlorides and have fewer horizontal surfaces onto which corrosive agents can deposit

Constructability considerations:

- Steel box girders have high torsional stiffness and can provide improved stability during erection
- Two span structure only requires the construction of one pier in the waterway

<u>Weaknesses – Concept 1</u>

- The steel structure is vulnerable to deterioration in the high salinity environment
- Steel girders require coating maintenance which will increase life cycle costs and asset management demands on the Department
- Long spans cause greater differential deflections between the box girders; this will require additional transverse deck reinforcing or intermediate transverse bracing, or a compromise of both and can be undesirable from a maintenance perspective
- Not supporting local fabricators. Steel girders will likely have to be fabricated outside of the province in Nova Scotia or New Brunswick
- Field splicing required
- If cracks were to develop in the deck, there is potential for moisture to enter the interior of the steel box girders and accelerate corrosion
- Steel superstructures are more susceptible to vibration
- Piers are inaccessible for future inspections and require divers
- Interior inspection of the girders will require confined space training
- Emergency services will lose access during construction.
- Utilities will need to be removed from the bridge.

<u>Opportunities – Concept 1</u>

- Continue to grow experience with steel bridge structures in NL

- Entice local steel fabricators to become certified and reduce costs on future box girder projects in the province
- Precast deck panels may be investigated to reduce differential deflection in girders during construction.

Threats – Concept 1

- Fluctuating steel prices
- Potential shipping restrictions
- Multiple deck pours provide more chances for construction schedule risks and overruns. This could be mitigated using precast deck panels
- Steel coating susceptible to deterioration due to chloride exposure and will require coating maintenance at years 15, 20, 27, 42, 47, 54, and 70.
- Scheduling risks of available equipment such as large cranes for erection
- If construction delays require over-wintering of erected girders, they will need to be covered to keep snow from accumulating inside the girders
- Inexperienced contractor could under bid work causing difficulties during construction
- Potential environmental contamination while constructing in water
- Strong currents could jeopardize construction efforts

9.1.2 Replacement Concept 2 – Three Span Prestressed Concrete Girder Bridge

Concept 2 is comprised of a three-span continuous structure constructed from prestressed concrete girders with a 225 mm thick reinforced concrete deck. Each span will be 40 m long. The girders will be supported on reinforced concrete abutments and two reinforced concrete piers. The total structural depth of this concept will not exceed the depth of the existing structure and therefore no realignment of the vertical grade is required.

CBCL presents three options for the prestressed concrete girder selection:

1. CPCI Girders – Five 1900mm deep
2. NEBT Girders – Five 1800mm deep
3. NU Girders – Four 1600mm deep

At the time of this report there are currently no pre-casting plants in NL with beds long enough to cast girders at this length. Therefore, the options will be to procure pretensioned girders from out of province or cast and post-tension the girders on site. Precast girders of similar length have been shipped to the province before. The replacement of Southeast Brook Bridge in Gros Morne National Park (Route 430) in 2016 shipped 39 m long NEBT prestressed girders from Nova Scotia. Alternatively, site casting is common bridge construction practice in NL due to limitations with local precast plants. CPCI girders lend themselves to site casting because they have simple angular geometry and do not require complicated formwork as other girder sections with rounded corners (i.e., NU and NEBT girders). It is our understanding that no contractor in NL has forms for casting NEBT or NU girders and to construct temporary forms for one project may prove cost prohibitive.

Both NEBT and NU girders offer longer span capabilities than CPCI girders for the same depth. The NU girders offer the longest spans of the I-type prestressed girders. The NU girder was developed by the University of Nebraska in response to industry demands due to span limitations with conventional I-type girders (i.e., AASHTO, CPCI, and NEBT). The NU girder has been used extensively in Alberta since 2001 on over 200 hundred bridges for span ranges from 20 m to 60 m[12].

Due to local contractor experience CBCL will proceed with CPCI girder selection for this concept. A feasibility study can be conducted during detailed design to investigate the merits of alternative prestressed girders. A post-tensioned girder will be the design approach taken by CBCL due to the ability to site cast/prestress with the added benefit of keeping all labour as local as possible.

9.1.2.1 SWOT Analysis – Superstructure Concept 2

Characteristics of this concept have been assembled in the following SWOT analysis:

Strengths – Concept 2

Maintenance, Repairs, and Inspection Features:

- Prestressed concrete girders require little maintenance
- Prestressed concrete offers a high degree of protection against corrosion of the reinforcing steel.
- Two less piers than the existing bridge

Constructability Considerations:

- Girders to be fabricated on-site which supports the local economy and eliminates the cost/complications with shipping
- Lower vibrations for this concrete system versus steel superstructure
- Transverse deck spans are small, reducing the need for additional transverse reinforcing

Weaknesses – Concept 2

- Two piers need to be constructed in the water. This will require consideration for temporary access and cofferdams to facilitate construction
- Piers are susceptible to debris and ice loads which will need to be included in design
- Piers are inaccessible for future inspections and require divers

[12] Alberta Transportation (2018). NU Girder Bridge Design and Detailing Manual. Vol 1. Version 1.

- Additional cost and schedule implications incurred by onsite post-tensioning and site casting operations.
- Emergency services will lose access during construction.
- Utilities will need to be removed from the bridge.

Opportunities – Concept 2

- Potential cost savings and other benefits of using alternative prestressed girders (i.e. NEBT and NU)
- Local suppliers/contractors could purchase an inventory of NEBT/NU girder forms for future projects
- Provides contractor flexibility in opting for on-site casting of girders or shop casting and shipping from a precast plant
- Girders are amenable to precast deck panels, potentially decreasing construction timeline if selected

Threats – Concept 2

- Inexperience of local contractors in post-tensioning girders of this depth and span
- Potential for low quality control when girders are post-tensioned on site
- Site casting of girders requires ideal weather conditions, which may impact project scheduling.
- Potential environmental contamination while constructing in water

9.1.3 Replacement Concept 3 – Four Span Prestressed Concrete Girder Bridge

Concept 3 features a four-span (30 m each) bridge constructed using five (5) CPCI 1400 prestressed concrete girders with a 225 mm thick reinforced concrete deck. The girders will be supported on reinforced concrete abutments and three piers. Based on the new vertical and horizontal alignment provided by the Department, the CPCI 1400 girders will be shallower than the existing bridge and therefore no anticipated change in the vertical alignment is required. It is anticipated that the concrete girders can be supported on integral abutments with end bearing piles driven to bedrock.

The girder lengths of 30 m allow the contractor the flexibility of either casting and post-tensioning on site or using a precast plant in St. John's or Nova Scotia and shipping to site. For this concept CBCL will assume that the girders will be precast/pretensioned to not exclude any potential contractors that do not have experience with site casting and post-tensioning. The Department's standard specifications, that will be issued for this project, include design stipulations for a contractor if they elect to offer a post-tensioned alternative following award.

9.1.3.1 SWOT Analysis – Superstructure Concept 3

Characteristics of this concept have been assembled in the following SWOT analysis:

<u>Strengths – Concept 3</u>

Maintenance, Repairs, and Inspection Features:

- Prestressed concrete girders require little maintenance
- Precast concrete is of superior quality and highly durable
- Precast concrete offers high degree of protection against corrosion of the reinforcing steel
- Increased hydraulic opening due to shallower depth

Constructability Considerations:

- No change in vertical alignment
- Potential for girders to be fabricated in NL (supporting local and eliminating the cost/complications with shipping)
- Lower vibrations for this concrete system versus steel superstructure
- Transverse deck spans are small, reducing the need for additional transverse reinforcing to meet special heavy truck requirements

<u>Weaknesses – Concept 3</u>

- Requires three piers to be constructed in water. This will require consideration for temporary access and cofferdams to facilitate construction
- Piers are susceptible to ice loads which will need to be included in design
- Piers are inaccessible for future inspections due to water level and will require divers
- Costs of transporting 30 m long girders from a precast plant to site
- Emergency services will lose access during construction.
- Utilities will need to be removed from the bridge.

<u>Opportunities – Concept 3</u>

- Provides contractor flexibility in opting for on-site casting of girders or shop casting and shipping from a precast plant; and
- Girders are amenable to precast deck panels, potentially decreasing construction timeline if selected.

<u>Threats – Concept 3</u>

- Inexperienced contractor could under bid work causing difficulties during construction
- Difficulty in controlling water during construction of the piers
- Potential environmental contamination while constructing in water

9.2 Abutment Selection

Each of the proposed superstructure concepts are amenable to integral or semi-integral abutments. Integral abutments are constructed without moveable transverse deck joints (expansion joints) or bearings at the piers and abutments. As the name implies, this type of foundation system is integral with the deck and supported by a single row of piles. The elimination of traditional deck joints and bearings is the main advantage in this type of foundation design as it significantly reduces maintenance efforts. Without joints, integral structures are subjected to additional thermal stresses which may result in cracking of the concrete. Also, due to the rigidity of an integral abutment structure, significant negative moments are developed at the end of the deck and top of the abutments. To mitigate these issues, integral bridges require additional reinforcing to resist the tensile stresses at these locations. Semi-Integral abutments, like integral abutments, are constructed without moveable deck joints in their superstructure but, unlike integral abutments, this type of foundation allows for the superstructure to move independently from the abutments with the use of bearings between the superstructure and rigidly supported abutments.

In lieu of geotechnical data, this assessment was based on the bedrock profile in the existing bridge drawings. The drawings show a shallow depth to bedrock below the proposed grade at each abutment which may not be sufficient to achieve the necessary pile lengths that promote the flexibility required for integral abutments. It is possible to core the bedrock to achieve the necessary pile length, however, the tributary lengths of each of the three proposed superstructure concepts may require a high number of piles to meet the end bearing load demand. An integral abutment is only as wide as the bridge deck therefore there is finite limit on the number piles that can 'fit' within an integral abutment because the piles need to be in a single row. Coring the bedrock requires a larger spacing between the piles and hence there may not be enough room to fit all the required piles. Trenching the bedrock, in lieu of coring, is an alternative, but not advisable in this situation due to the complexities of trenching rock below the waterline. It is for these reasons that CBCL recommends that the Marystown Harbour bridge replacement be considered for semi-integral abutments at this stage given the limited geotechnical information. This justification should be re-visited with geotechnical data on the load bearing capacity of the bedrock during detailed design. The abutments should still be constructed on piles because a piled foundation reduces the amount of reinforced concrete required for the abutments and avoids having to construct in the water.

9.3 Pier Configuration Selection

Each of the proposed superstructure concepts require intermediate piers to be constructed in the channel opening. CBCL proposes that the pier be constructed as a monolithic wall. This type of pier is easier to construct due to its simple geometry which is important in this challenging environment. The wall will be a constant thickness that is wide enough to mount the bridge structure directly on. Transverse loads will be transferred from shear blocks bearing laterally on diaphragms in the superstructure. A rounded bull nose is recommended for the shape of the wall ends. The bullnose shape has performed very well on other bridges subjected to ice forces (Figure 9-2) whereas the armoured diamond tip is more prone to concrete spalling as the plate inevitably begins to abrade and corrode. Often monolithic wall concepts are abandoned for thinner walls and wider cap beams that save on material costs. However, this concept is not practical in this location due to the limited space available for the cap beam above high water.

The pier wall will be subjected to chlorides from seawater saturation, freeze thaw cycles, and severe erosion forces from tidal fluctuations and ice abrasion. The amount of deterioration evident on the existing piers can attest to how aggressive this environment is on concrete structures. Mitigating maintenance in the long-term is a priority for the Department, therefore CBCL recommends that a concrete mix design be developed for this environment that is beyond the Departments standard concrete specification and to consider the use of GFRP reinforcing in the pier wall. GFRP does not corrode which will mitigate future concrete spalling. GFRP is much less expensive than stainless steel reinforcing and does not present any galvanic corrosion issues when mixed with galvanized/black reinforcing steel.

Figure 9-2 – Condition of rounded bullnose on existing pier in service for over 50 years

9.4 Recommended Replacement Option

Concept 2 is CBCL's recommended solution for replacing this structure. This solution consists of a three-span continuous prestressed concrete girder on two piers and semi-integral abutments. This structure is constructed from durable materials for this harsh environment and only requires two piers to be constructed in the water. A concept drawing for this option is included in Appendix R.

Although not a part of this scope, CBCL recommends that the Department consider shortening the waterway length through means of a causeway to reduce the span of the new bridge for potential cost savings. These details would need to be explored during detailed design.

Chapter 10 Evaluation of Rehabilitation vs Replacement

The decision matrix to rehabilitate Marystown Harbour Bridge or replace it considers the service life, life cycle costs, and the strengths, weaknesses, opportunities, and threats of each option.

10.1 Service Life Comparison

The rehabilitation strategy for Marystown Harbour Bridge is designed to extend the service life of the bridge for another twenty-seven (27) years. After this time, it is anticipated that more elements/components may begin deteriorating and another major rehabilitation effort will be required. The recommended replacement option is designed (and life cycle costed) for a service life of seventy-five (75) years. It is anticipated that only minor concrete repair works and bearing replacement will be required during this life span.

10.2 Costs Consideration Comparison

The cost of rehabilitating this structure is comparable to replacing the structure with a similar span and width. The cost of replacing the bridge is approximately 2% more than the cost of rehabilitating the bridge to extend its service life for another twenty-seven (27) years.

10.3 SWOT Analysis Comparison

A SWOT analysis comparison between rehabilitating or replacing Marystown Harbour Bridge is summarized in Table 10-1.

Table 10-1: SWOT Analysis Comparison – Rehab vs Replacement

Rehabilitation	Replacement
Strengths	
▶ No significant earthworks or realignment required ▶ Lower carbon footprint by re-using original structure ▶ Marginally lower capital costs	▶ Longer service life expectancy ▶ Less long-term maintenance requirements ▶ No expansion joints required ▶ Less in water elements ▶ Greater reliability in structural capacity

Weaknesses	
- Service life for only 27 years - Increased dead load may require additional piles in P1 - Major disruption to traffic. Bridge may need to be temporarily closed to carry out repairs - Extensive amount of surface preparation required to strengthen and recoat bridge - Public distaste for rehabilitating a bridge that was deemed unfit for service.	- Moderate disruption to traffic during construction - Load restrictions will be present on existing bridge until construction is finished. Subsequent re-inspection will be required to monitor corrosion on the existing bridge. - Emergency services will lose access during construction. - Utilities will need to be removed from the bridge.
Opportunities	
- Preserve a structure which has a cultural significance for the town/region. - Develop a method for rehabilitating concrete piers in a marine environment	- Girders are amenable to precast deck panels, potentially decreasing construction timeline if selected - Increase hydraulic capacity - Increase navigational height - Widen roadway / add traffic lanes - Bridge can be shortened by extending a causeway.
Threats	
- ASR may still be active which can jeopardize the durability of the repairs. - Potential environmental containment during removal of hazardous paint - Existing structure may not meet Department hydraulic capacity standards - Inclement weather can threaten project schedule - Strong currents will be challenging to install pier formwork - Steel testing show significant variability in existing steel indicating low quality control during original construction. - Rehabilitation of the steel components may introduce damage due to extensive welding or other repair activities.	- Inclement weather can threaten project schedule - Strong currents will be challenging to install cofferdams - Disruption to marine life during construction

10.4 Recommendations

The cost savings of rehabilitating Marystown harbour Bridge versus replacement are negligible and far outweighed by the increased durability and reliability of a new structure. A new bridge will provide the department with a lower maintenance structure and will gain an additional forty-eight (48) years of service life for the bridge. CBCL recommends that the Department forego the risks and lower service life associated with rehabilitation and plan for the immediate replacement of this structure.

Chapter 11 Conclusions & Recommendations

This report provided a detailed assessment of Marystown Harbour Bridge. The report presented the findings from a comprehensive visual assessment, a thorough structural analysis including a live load evaluation, seismic evaluation, and gusset plate analysis, and summarized in-situ material testing performed on the bridge. The information yielded from these tasks were used to provide recommendations to enhance the service life of the structure.

The major findings that were concluded from the site inspection include:

- Concrete deterioration is widespread in both abutments, wingwalls, and deck topping.
- All piers are very severely eroded at the waterline and spalled/delaminated at the nosing. The pier shaft concrete is not durable for this environment.
- Medium to severe corrosion on the main girder flanges.
- The structural inadequacy of the traffic barrier.

The major findings that were concluded from the live load evaluation, fatigue assessment, seismic analysis, and splice plate analysis include:

- Main girders do not meet the load evaluation demands of the CHBDC. **CBCL recommends the bridge be closed to traffic**. Specific emergency vehicles may safely cross the bridge under the restrictions described in this report.
- The piers do not technically comply with CHBDC or CSA A23.3 but do appear to have sufficient axial and bearing capacity to permit unrestricted traffic access.
- The following members failed a fatigue assessment: top and bottom flanges of the main girders in the continuous and swing spans and the floor beams in both spans.
- The bridge was theoretically deemed to perform satisfactorily based on the parameters assumed in the seismic analysis.
- The splice plates on the continuous span, for both the east and west girders, were over-utilized in the failure modes of yielding of gross section and fatigue.

The major findings that were concluded from the material testing include:

- The bridge deck concrete is in fair to poor condition.
    - The bridge deck concrete was measured to be sufficiently air entrained.

   - Chloride ion concentrations have been measured to exceed those required to promote corrosion of the embedded reinforcing steel at all locations tested.
   - No evidence of alkali-silica reactivity (ASR) was observed.
- The substructure concrete is in very poor condition.
   - Compressive strengths ranging from 34.8 to 50.7 MPa.
   - Entrained air void network has been measured to be substandard. The significant mass loss of substructure elements is likely due to a combination of deterioration including cyclic freezing and thawing.
   - Chloride ion concentrations have been measured to exceed those required to promote corrosion of the embedded reinforcing steel at all locations tested.
   - Evidence of alkali-silica reactivity (ASR) was observed both by visual inspection and confirmed with Cornel gel fluorescence testing.
- No fatigue cracks were present in the splice plates that were tested.
- The paint contains very high levels of lead.
- Steel coupon testing shows that strengths vary throughout the bridge resulting in a yield strength used in analysis less than those specified on the original construction drawings.

This bridge will require an extensive rehabilitation to extend its service life to 2050. Items considered for rehabilitation included: traffic barrier replacement, abutment and deck concrete repairs, new approaches, main girder strengthening, bearing replacement, secondary steel replacement, and concrete encapsulation of the piers.

Three viable replacement options were considered. The options are characterized by their superstructure and include:

- Concept 1: Two span steel box girder
- Concept 2: Three span prestressed concrete girder
- Concept 3: Four span prestress concrete girder

Concept 2 was selected by CBCL due to the more favorable SWOT analysis when compared to the other two replacement options.

A comparison analysis between rehabilitation and replacement pointed strongly in favor of replacement. A new structure is approximately the same costs of rehabilitation and will provide a more reliable and durable structure for double the service life. For these reasons CBCL recommends that the Department should consider replacement of this structure rather than rehabilitation.

Chapter 12 Closure

We trust that this report provides the information required by the Department to fully assess the condition of Marystown Harbour Bridge and the information contained herein is sufficient to make informed decisions regarding the feasible life extension of this bridge.

Please do not hesitate to contact the undersigned if you have any questions or concerns.

Mitchell Warren

Prepared by:
Mitchell Warren, B.Eng., EIT
Junior Bridge Engineer

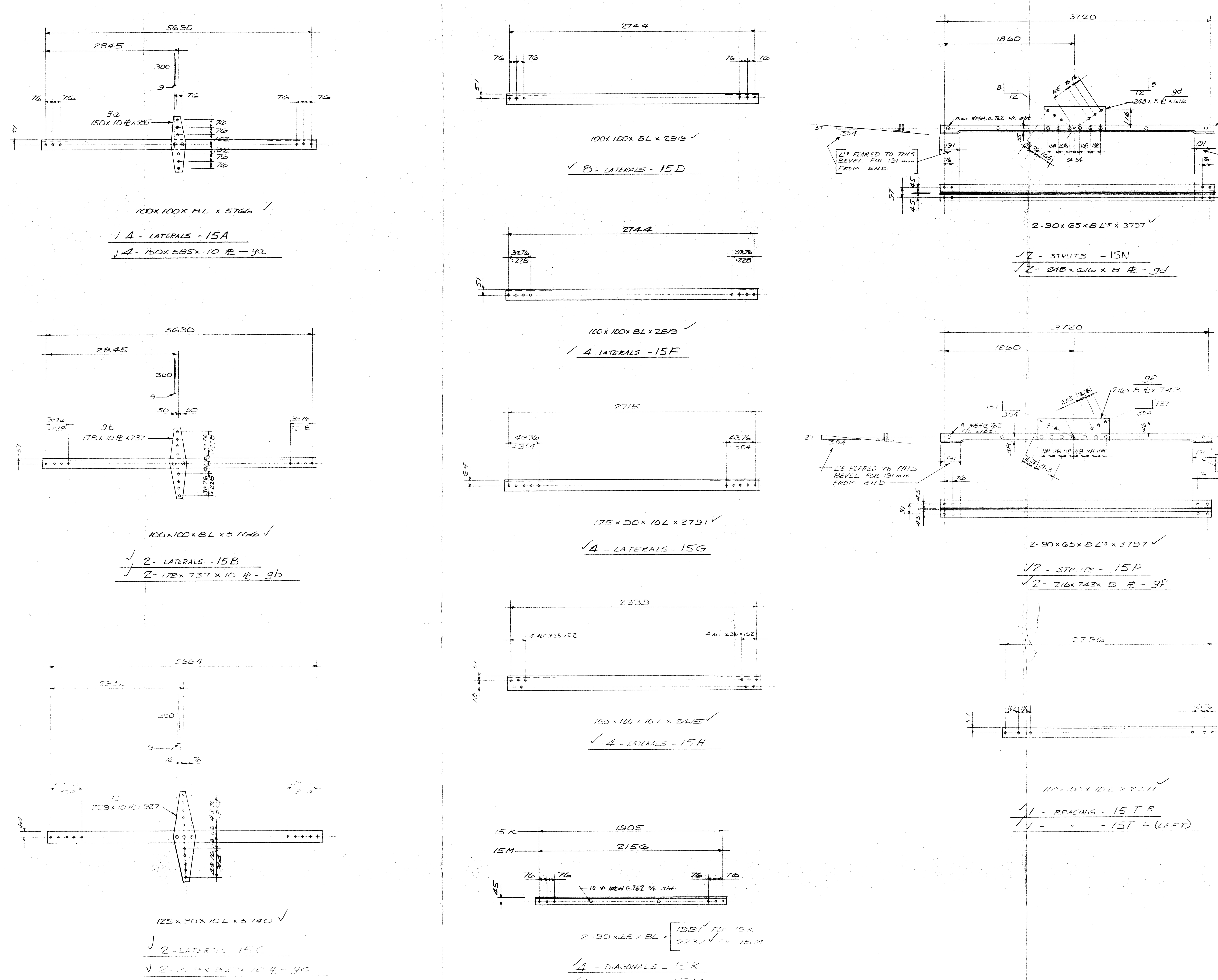
Colin Jim

Reviewed by:
Colin Jim, P.Eng.
Manager of Bridge Department

APPENDIX A

Existing Drawings


South
North
Buffer Columns
Rock filled 12'x12' Cribs
130'-0 Swing Span
65-0
65-0
80-0
100-0
80-0
Sta. 9+00
Sta. 8+00
Sta. 7+00
Sta. 6+00
El. 105.0
El. 105.0
El. 95.5
El. 95.5
H.W.L. El. 91.5
Extreme Low Tide El. 84.5
50'-0 Clear
Exp.
Fixed
Exp.
Exp.
Gravel with Boulders
Gravel with Boulders
South Abutment.
Pivot Pier
Pier No3
Pier No2
Pier No1
North Abutment.
Elevation
Scale 1in.=20ft.
Marystown Harbour Bridge, L=392', W=25'.
South Abutment
Scale 1in.=8ft.
R=66'-4½"
Pivot Pier
Scale 1in.=8 feet.
El. 95.5
El. 73.0
El. 63.0
21'-0"
21'-0"
25'-0"
North Abutment
Scale 1in.=8ft.
El. 105.0
El. 95.5
El. 95.5
Dowels 7/8"x5', 12" o.c.
Dowels 1¼"x8', 18" o.c.
Pier No3. Scale 1in.=8ft.
Pier No2. Scale 1in.=8ft.
Pier No1. Scale 1in.=8ft.
General Notes:
1. Concrete in substructure to have 3000 PSI compressive strength in 28 days
2. Not less than 1in. bevels shall be placed at the angles of form to round the edges
3. All piles to be H Steel piles BP10, 42lbs/ft.
4. All piles to be driven to refusal.
5. All work shall be performed in accordance with American Association of State Highway Officials Specifications for Highway Bridges (1953)
MARYSTOWN HARBOUR BRIDGE
Substructure with Gravity Piers
SCALE: As shown
DATE: Apr. 1956
PROVINCE of NEWFOUNDLAND
DEPARTMENT of HIGHWAYS
BRIDGE OFFICE
DRW. 1ar
Revised Jan. 1958, after completion of construction of substructure



APPROVED AS NOTED

This drawing is approved as to general arrangement subject to corrections noted thereon. This approval does not release the Contractor from responsibility for conformity to specifications and correct details. The Contractor is responsible for checking all dimensions.

June 28/91

Date

Signed By

Shop drawings are approved on the Condition that following additional information & revisions are to be made:

1) Item No.3 in your notes on Sheet 1 of 3 shall be deleted.

2) Add to Notes: Welding shall be done in accordance to CSA W59 & W47.1.

3) On Sheet 2 of 3 and Sheet 3 of 3, Please identify your weld sizes for the gusset plates and for the new Strut systems. Weld sizes Shall be Submitted prior to any welding.

NOTES:

1. BOLTS SHALL BE 22mm ϕ.
2. BOLT HOLES SHALL BE 24 mm ϕ.
3. ALTERNATELY GUSSET ℞ MAY BE WELDED TO LATERALS.
4. MATERIAL GRADE 300W
5. FABRICATION IN ACCORDANCE WITH SECTION 910 - DOWST SPECIFICATION BOOK.
6. BOLTS GALVANIZED A325M

NO.	DESCRIPTION	DATE	BY

REVISIONS

A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

ARCHITECT / ENGINEER / CONSULTANT

TRIDENT CONSTR LTD.
19 DUNDEE AVE.
DONOVANS IND. PARK

PROJECT

REHABILITATION OF
MARYSTOWN CANNING BRIDGE

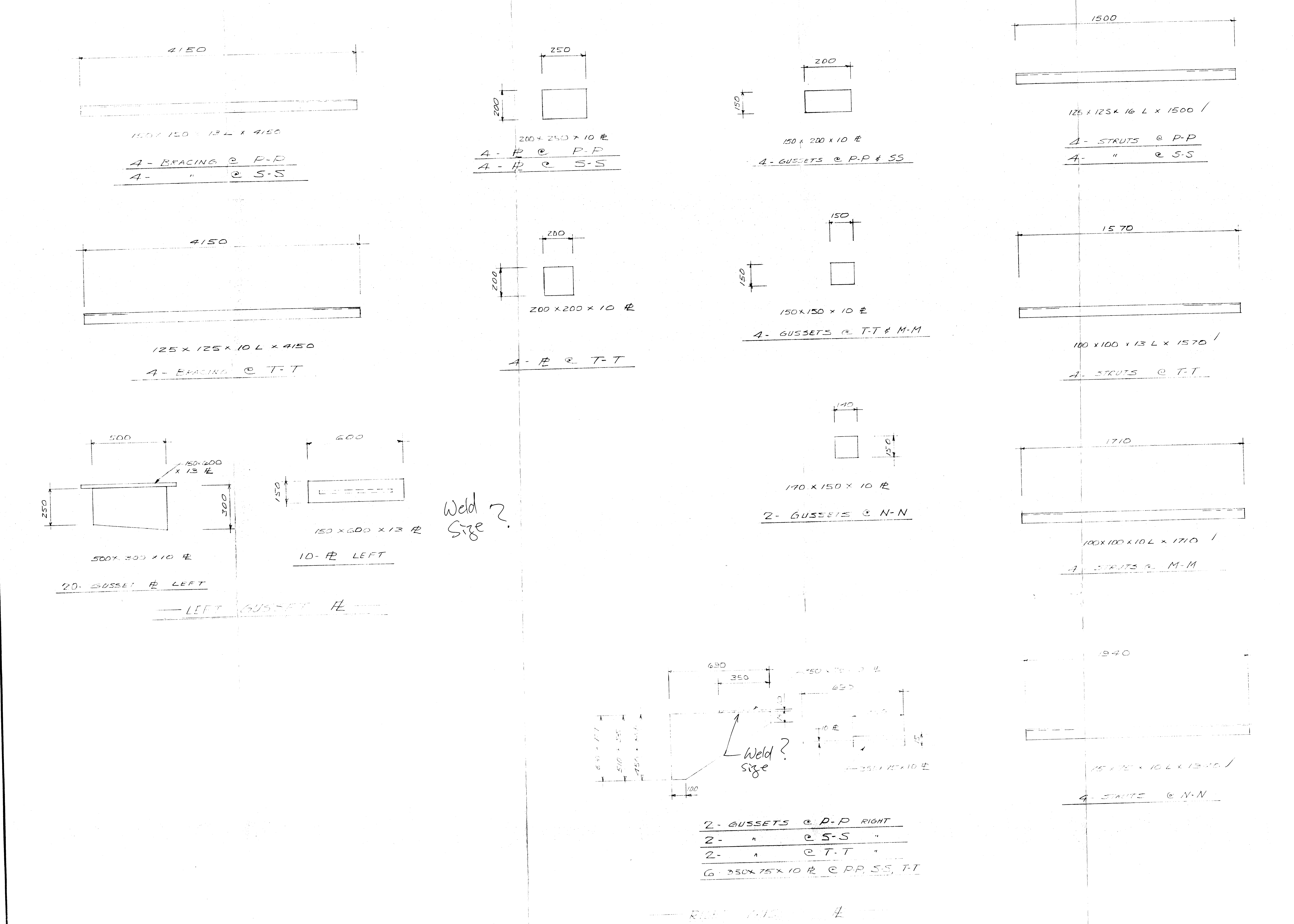
DRAWING

SWING SPAN
BRACING & BRACE FRAMES

DRAWN BY	CHECKED BY	APPROVED BY	
PROJECT NO. 7-91 RSR	DWG. FILE NO.	FILE NO.	
DATE 91-06-18	SCALE NTS	DRAWING NO 1 OF 3	REV.

0mm 100mm 200mm 300mm

A0 841mm x 1189mm



APPROVED AS NOTED
This drawing is approved as to general arrangement subject to corrections noted thereon. This approval does not release the Contractor from responsibility for conformity to specifications and correct details. The Contractor is responsible for checking all dimensions.

June 28/91
Date

Signed By

NO.	DESCRIPTION	DATE	BY

REVISIONS

A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

REGISTERED PROFESSIONAL ENGINEER
DERRICK IVANY SPRACKLIN

ARCHITECT / ENGINEER / CONSULTANT
TRIDENT CONSTR. LTD
19 DUNDEE AVE
DONOVANS IND. PARK

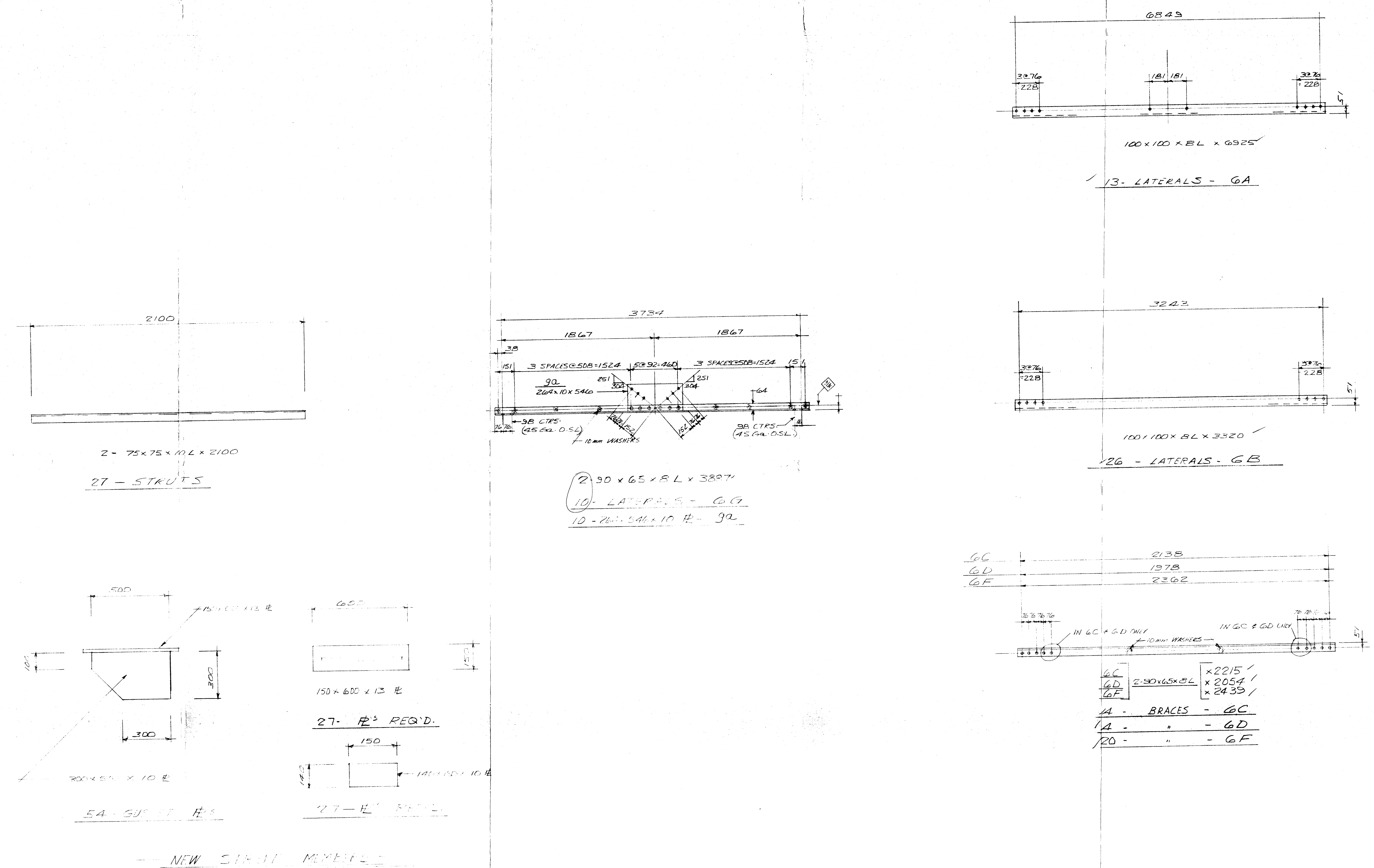
PROJECT
REHABILITATION OF
MARYSTOWN CANNING BRIDGE

DRAWING
SWING SPAN
NEW STRUT MEMBERS &
ADDITIONAL BRACING

DRAWN BY	CHECKED BY	APPROVED BY	
PROJECT NO. 7-91 PSK	DWG FILE NO.	FILE NO.	
DATE 91-06-13	SCALE NTS	DRAWING NO. 2 OF 3	REV.

0 mm 100 mm 200 mm 300 mm

A0 841 mm x 1189 mm



APPROVED AS NOTED

This drawing is approved as to general arrangement subject to corrections noted thereon. This approval does not release the Contractor from responsibility for conformity to specifications and correct details. The Contractor is responsible for checking all dimensions.

June 28/91

Date Signed By

NO.	DESCRIPTION	DATE	BY

REVISIONS

A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

ARCHITECT / ENGINEER / CONSULTANT

TRIDENT CONSTR. LTD
19 DUNDEE AVE.
DONOVANS IND PARK

PROJECT

REHABILITATION OF
MARYSTOWN CANNING BRIDGE

DRAWING

CONTINUOUS GIRDER
NEW STRUTS &
BRACING

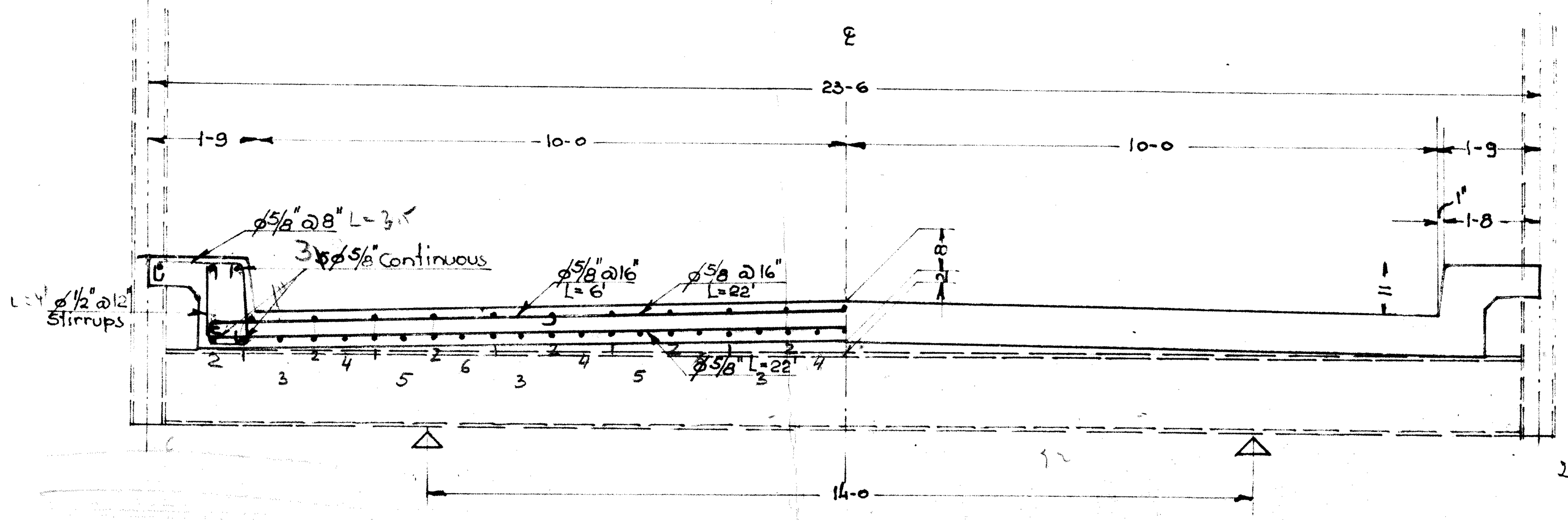
DRAWN BY	CHECKED BY	APPROVED BY	
PROJECT NO. 7-91 PSR	DWG. FILE NO.	FILE NO.	
DATE 91-06-18	SCALE NTS	DRAWING NO. 3 OF 3	REV.

0 mm 100 mm 200 mm 300 mm

A0 841mm x 1189mm

LONGITUDINAL SECTION - 80 FT. SPAN

LONGITUDINAL SECTION - 100 FT. SPAN



TYPICAL CROSS SECTION

PLACE BARS IN FOLLOWING SEQUENCES :
1,3,2,4,1,5,2,6,1,3,2,4,... ETC.

R. STEEL FOR 2×80' and 100' SECTIONS

	No	ø			Placing	To be ordered		Scraps	
80'	1,2,	3/4"	(13 1/3 + 40' + 34) × 2 × 22	=	3843	15+44+44 = 103 bars	4120'	13 1/3 + 14 × 6' = 277'	✓
100'	1+2	3/4"	[(2×32+40)+2×40)+24] × 11	=	2288	22+11+22+11 = 66 "	2,640'	22×8 + 11×16 = 352'	✓
80	3,5	3/4"	(37+38+26) × 2 × 11	=	2222	22+22+22 = 66	2,640'	22×3'+22×2'+22×14 = 418	✓
100'	3,5	3/4"	(35+38+38+13 1/3) × 11	=	1368	11+11+11+4 = 37	1480'	11×5+22×2+13 1/3 = 112	✓
80	4,6	3/4"	(35+36+25) × 2 × 10	=	1920	20+20+20 = 60	2400'	20×5+20×4+20×15 = 480	✓
100	4,6	3/4"	(33+36+36+13) × 10	=	1180	10+10+10+4 = 34	1360'	10×7'+20×4'+27+34 180	✓
		3/4	3'×600	=	1800				
					14621	ø3/4 - 366	14640'		
2×80+100	Distr.	5/8	22'(156+92)	=	5456	248			
	"	5/8	22'(126+76)	=	4444	202			
	"	5/8	6'(248+150)	=	2,388	450 bs - 18000'			

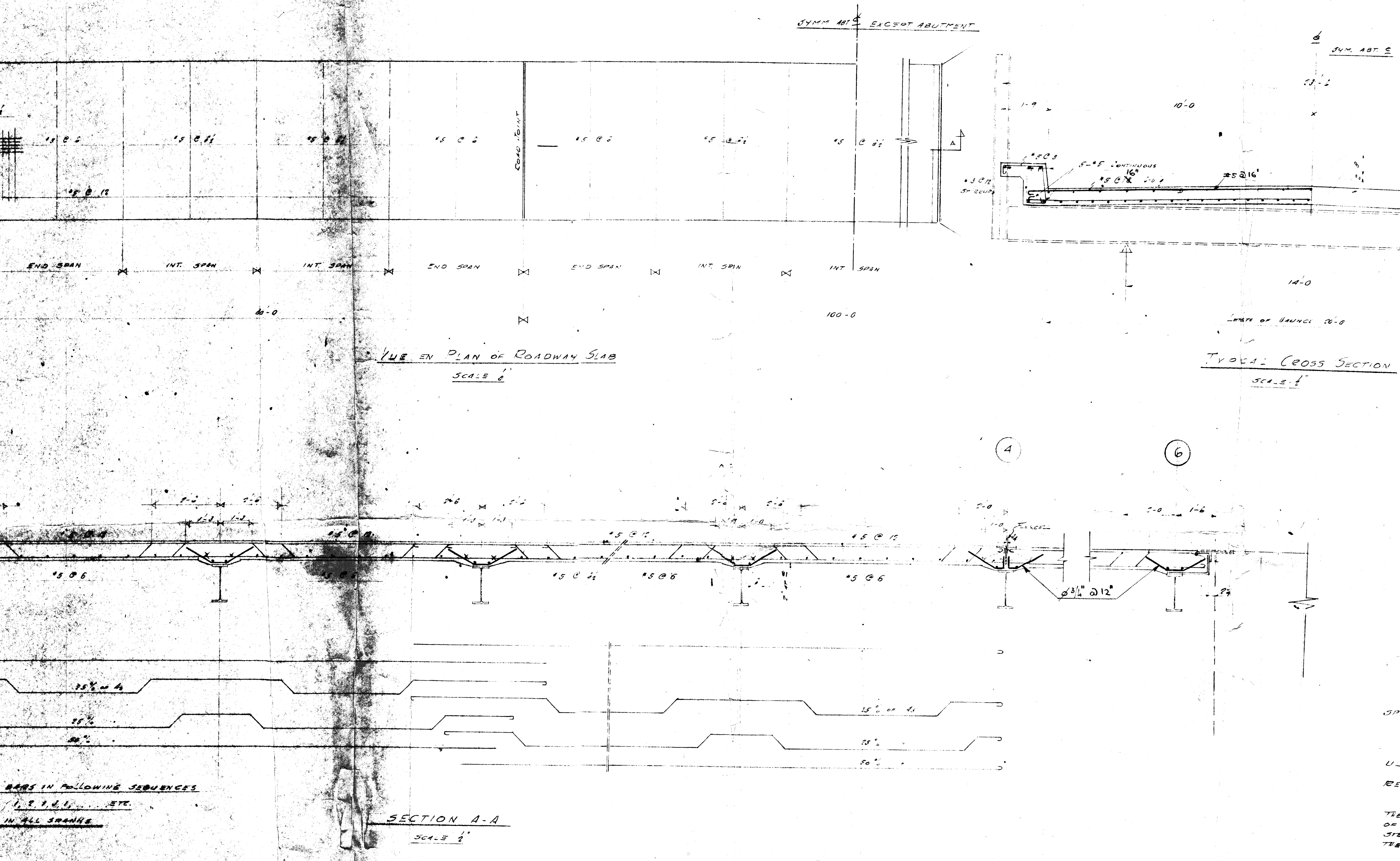
MARYSTOWN HARBOUR BRIDGE

PROPOSED SLAB REINFORCING FOR 80 FT. & 100 FT. SPANS

SCALE : 1/2" = 1' | DATE : Mar, 1959

PROVINCE of NEWFOUNDLAND
DEPARTMENT of HIGHWAYS
BRIDGE OFFICE

FILE No RD2/C10/1



GENERAL NOTES

SPECIFICATIONS: FOR STEEL HIGHWAY

FOR REINFORCED C
C. A

ULTIMATE STRENGTH OF CONCRETE

REINFORCEMENT: INTERMEDIATE G

THE COVERING MEASURED FROM THE S
OF THE CONCRETE TO THE FACE OF AN
STEEL SHALL BE 2" EXCEPT IN SLA
THE COVERING SHALL BE 1"

WORK THIS DRAWING WITH DRWGS E1 &
CONT 1-8737

DATE	REVISION	MADE BY

DOMINION BRIDGE CO., L
LACHINE, QUE.

MARYSTOWN HARBOUR B

DEPARTMENT OF PUBLIC

PROVINCE OF NEWFOUN

ROADWAY SLAB FOR APPROACH

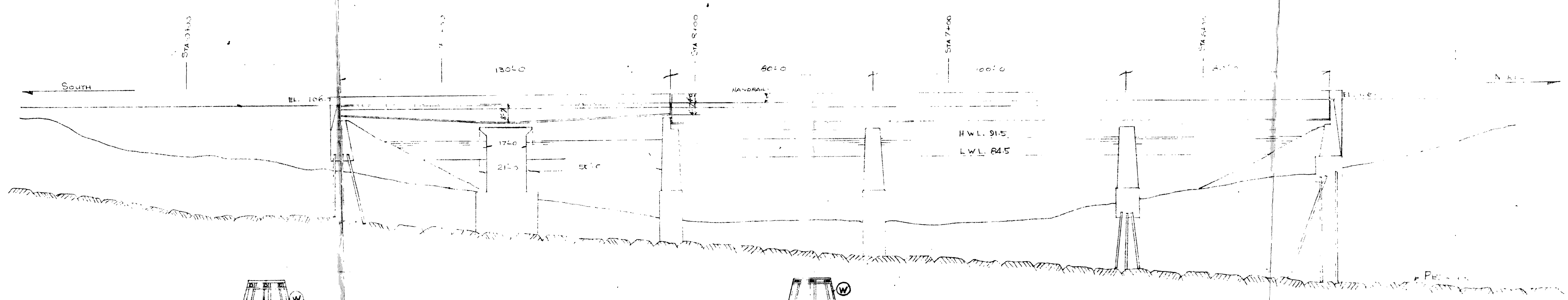
DEPARTMENT STRUCTURAL DESIGN SCALE

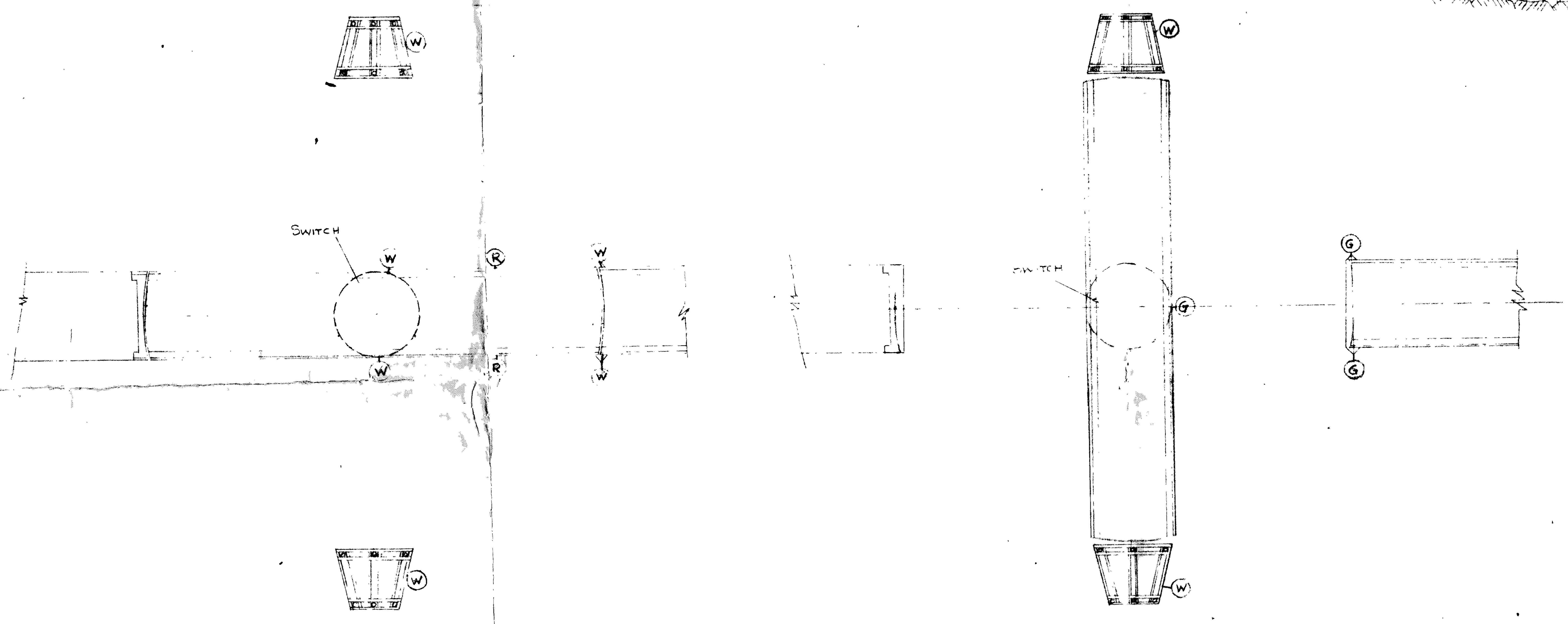
MADE BY A. S. DATE

DRAWING No. D-4-D FILE No.

MADE IN CANADA

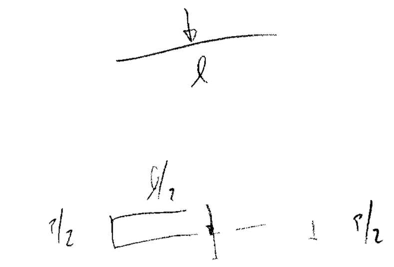
MARYSTOWN HARBOUR BRIDGE	
PROFESSIONAL DIVING CONTRACTORS LTD.	
DRAWN BY: RG	CHECKED BY:
SCALE: AS SHOWN	DATE: 83.11.07





PLAN BRIDGE CLOSED
SCALE 1" = 20'-0

PLAN BRIDGE OPEN
SCALE 1" = 20'-0



NOTES

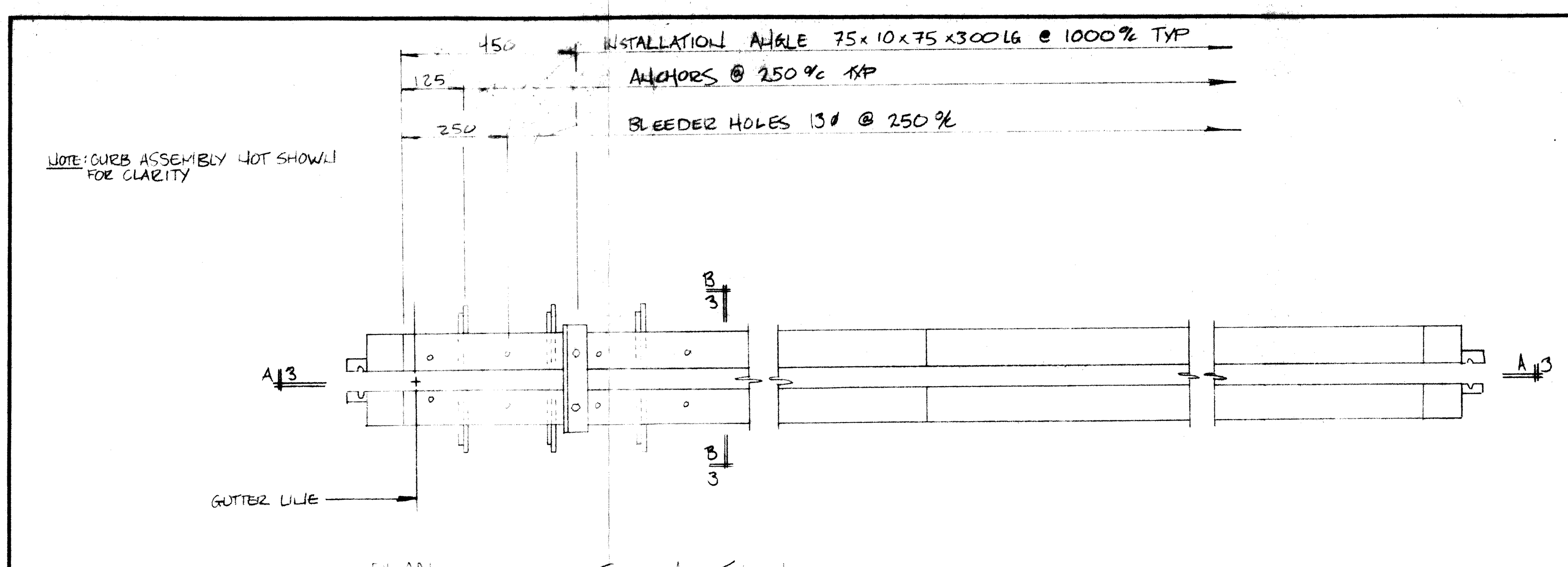
1. (R) RED — 100 WATT ~~CANDLE POWER~~
   (W) WHITE — 60 WATT ~~CANDLE POWER~~
   (G) GREEN — 100 WATT ~~CANDLE POWER~~
2. [illegible] MUST BE CAPABLE OF BEING SWITCHED FROM GENERATOR TO TOWN ELECTRICAL SUPPLY
3. LIGHT SWITCH MUST BE LOCATED ON THE BRIDGE AS SHOWN

MARYSTOWN HARBOUR BRIDGE

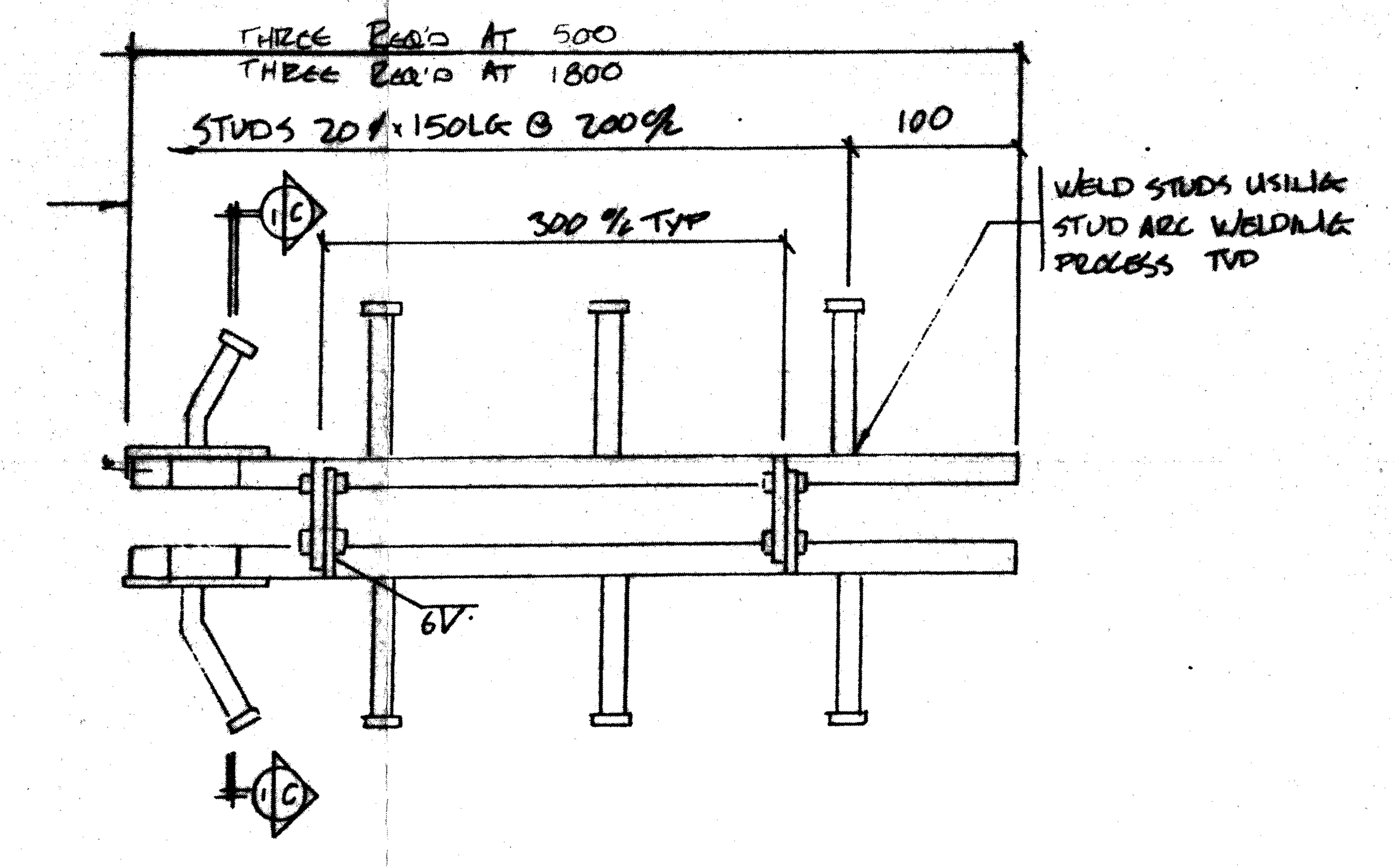
LIGHTING

SCALE : AS SHOWN | DATE : [illegible]

PROVINCE of NEWFOUNDLAND
DEPARTMENT of HIGHWAYS
BRIDGE DEPT.

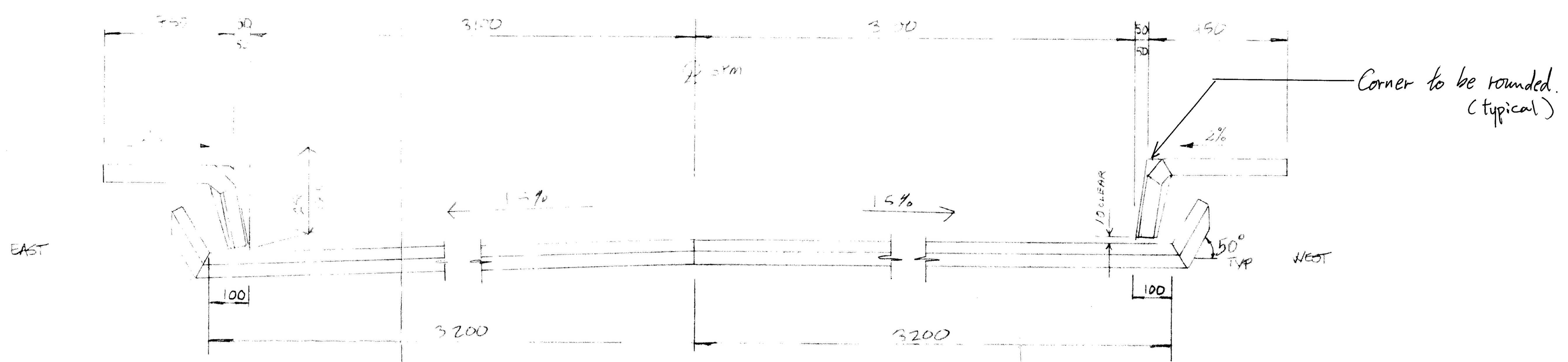


PLAN

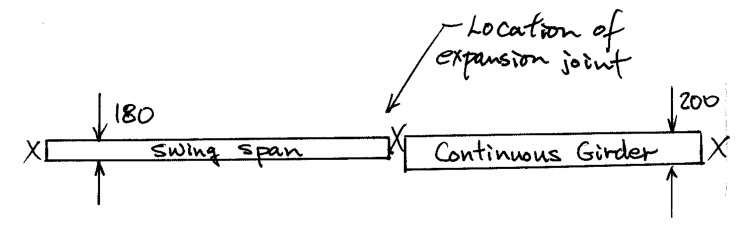


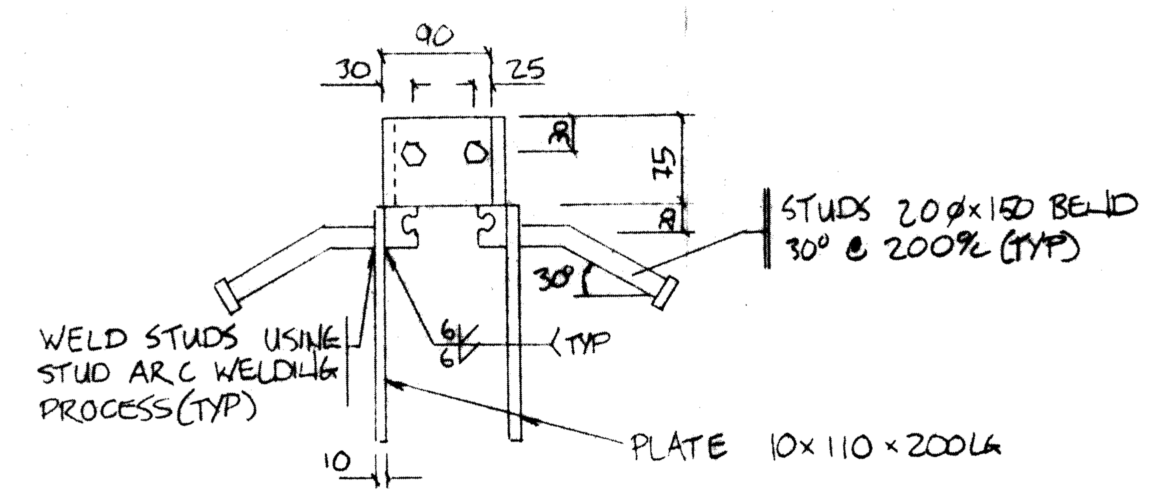
PLAN OF CURB ASSEMBLY
1:5

GAP ADJUSTMENT TABLE °C							
LOCATION CANNING BRIDGE	0°	5°	10°	15°	20°	25°	30°
NORTH	68	65	63	60	58	55	53
PIER	68	65	63	60	58	55	53
SOUTH	68	65	63	60	58	55	53

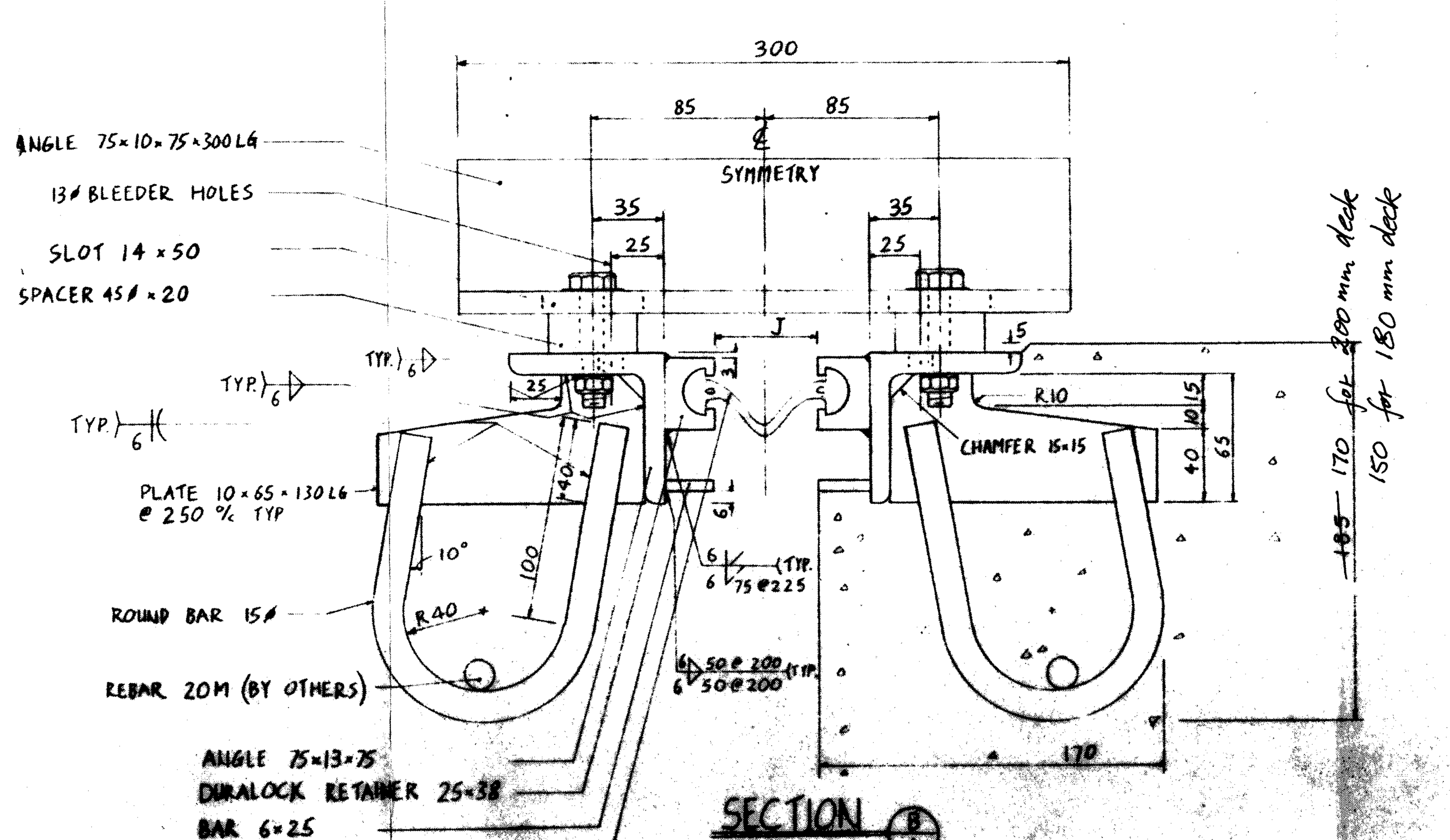


SECTION A

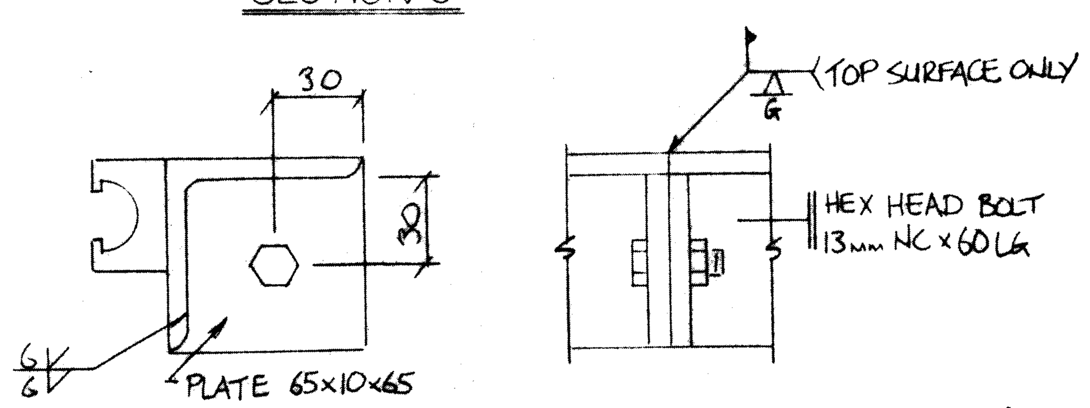




SECTION C



SECTION B
1:2



SPLICE DETAIL B

APPROVED AS NOTED

This drawing is approved as to general arrangement subject to corrections noted thereon. This approval does not release the Contractor from responsibility for conformity to specifications and correct details. The Contractor is responsible for checking all dimensions.

July 4/91
Date

Signed By

Shop drawings ~~are~~ is approved on the condition that the following revisions are to be made:

1. Corners on the top of the curb & sidewalk ~~should~~ shall be rounded.
2. The depth of the expansion joint shall be revised to accommodate the different thickness of concrete deck.

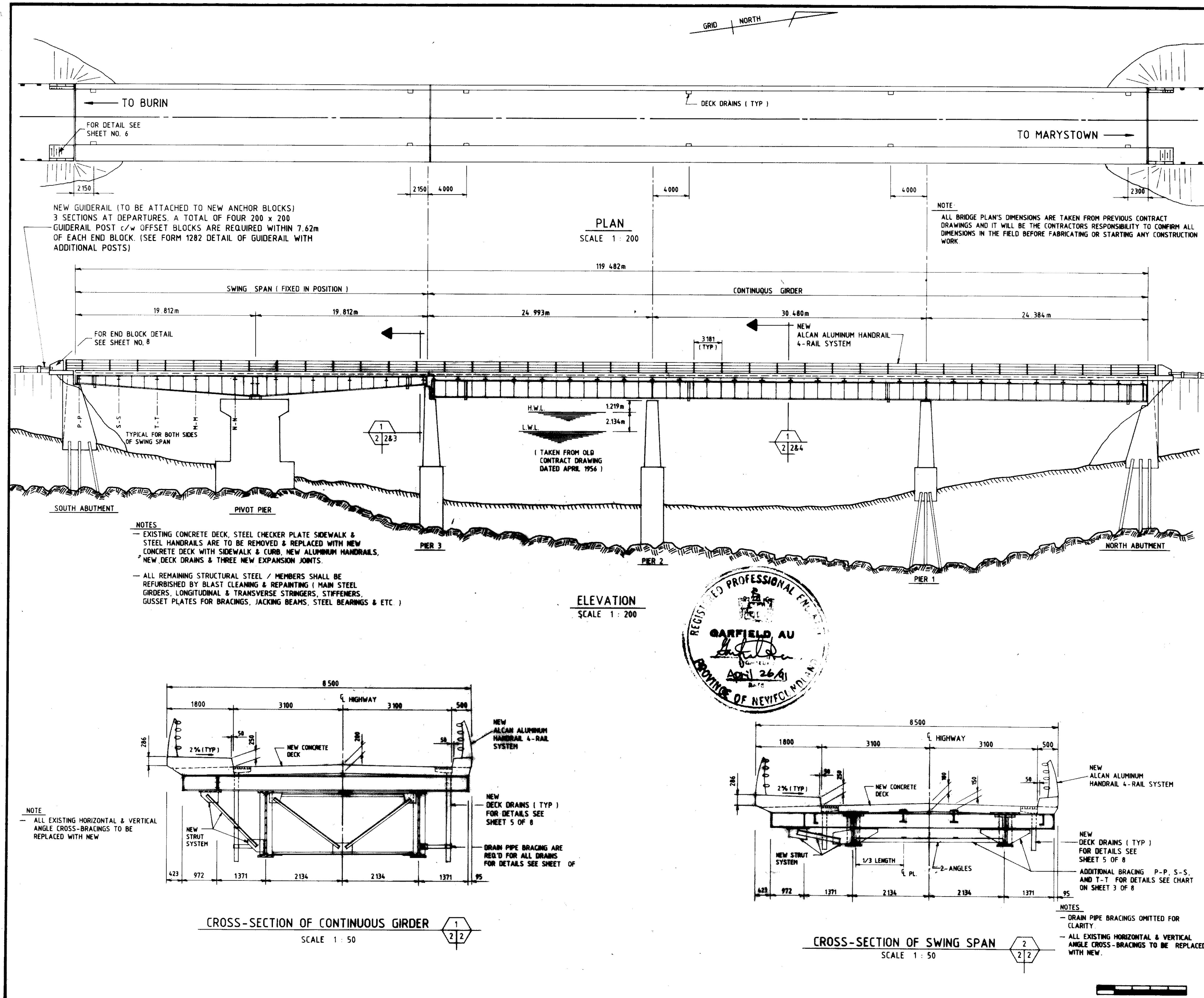
NOTES

1. STEEL SHALL BE IN ACCORDANCE WITH CSA STANDARD CAN 3. G40.21-M81 GRADE 300W
2. JOINT ASSEMBLY SHALL BE INSTALLED TO MATCH GRADE OF ROAD SURFACE.
3. TEMPORARY INSTALLATION PLATES AND ANGLES TO BE REMOVED AFTER CONCRETE HAS SET.
4. ALL DIMENSIONS IN MILLEMETRES.
5. SEAL TO BE INSTALLED IN STAGE 1 AND ALL CURBS IN SHOP.
6. JOINT TO BE PRESET FOR 10°C.
7. DESIGN TO CSA-S6-M88 (MS250 LIVE LOAD).
8. DURALOCK RETAINER TO BE COATED WITH CORROSION PROTECTION AGENT.
9. NEOPRENE SEAL TO ASTM D2628.
10. WELDING TO CSA W59 AND RELATED STANDARDS.

JOB NO.	STELENCO LTD.	1 OF 1
CHKD. BY	SINGLE SEAL JOINT	
DRN. BY W. C[illegible]	CANNING BRIDGE	
DATE JUNE 91		
SCALE		
DWG. NO. J-91-47	PLAN AND DETAIL	

REVISION 1 JUNE 91

FOR TENDER ONLY



— NOTES —

DO NOT SCALE DRAWING

1. DESIGN LOAD IS HS20-44.
2. NO ASPHALTING TO BE PERMITTED OVER THE CONCRETE DECK.
3. CONCRETE IN DECK SHALL BE 35 MPa IN 28 DAYS.
   AIR: 5-8%
   SLUMP: 40-60 mm
   FINISH: CLASS 6
4. MINIMUM CONCRETE COVER TO REINFORCING STEEL:
   (a) TOP & SIDE OF DECK 50 mm
   (b) BOTTOM OF DECK 30 mm
5. ALL WORK SHALL BE IN ACCORDANCE WITH CAN/CSA-S6-88 NATIONAL STANDARD "DESIGN OF HIGHWAY BRIDGES" AND CAN 3-A23.1-M77 "CONCRETE MATERIALS AND METHODS OF CONSTRUCTION".
6. REINFORCING STEEL: HARD GRADE, YP 400 MPa.
7. MINIMUM LAP LENGTHS SHALL BE AS FOLLOWS (mm):

15M	630
20M	800
25M	1300
30M	1800
35M	2600

8. MINIMUM DIAMETER OF BENDING (mm):

15M	95
20M	120
25M	150
30M	240
35M	360

CONT ------- SHEET 3

NO.	BY	DESCRIPTION	DATE

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

TITLE:
GENERAL ARRANGEMENTS

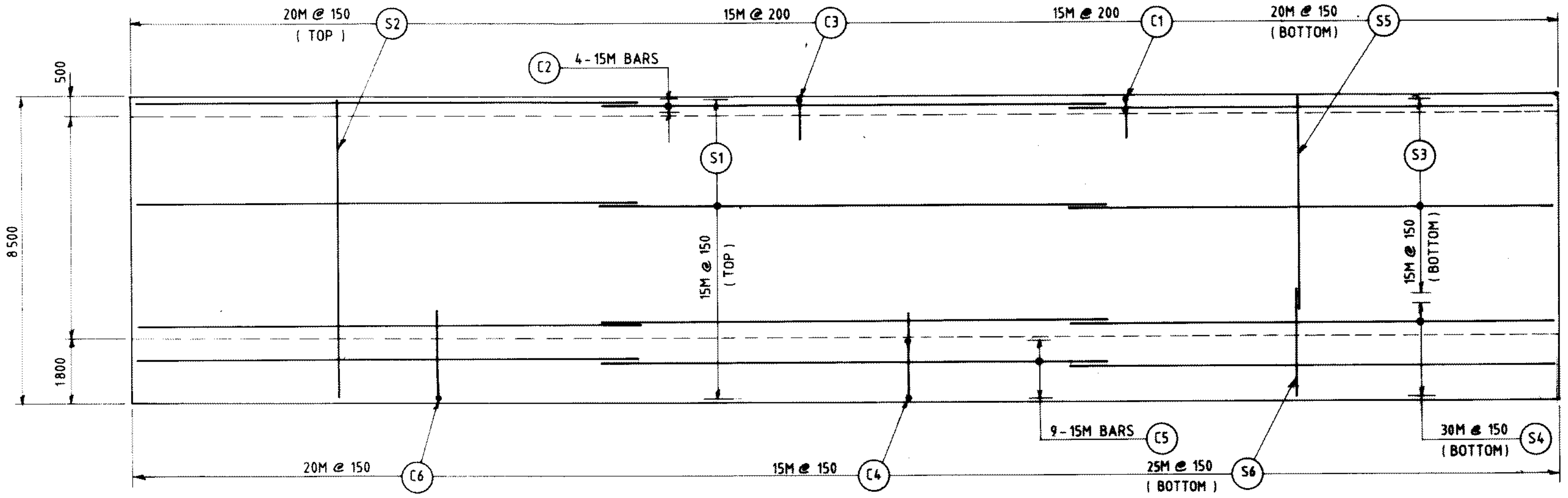
SCALE: AS SHOWN	DATE: 91/01/09	DRAWN BY: H Jones
DWG. CHK'D. BY: G. Au	DESIGNED BY: G. Au	DES. CHK'D BY:
APPROVED BY: R.D.L.	DWG. NO.	FILE NO.

PROJECT NO.
7-91PSR

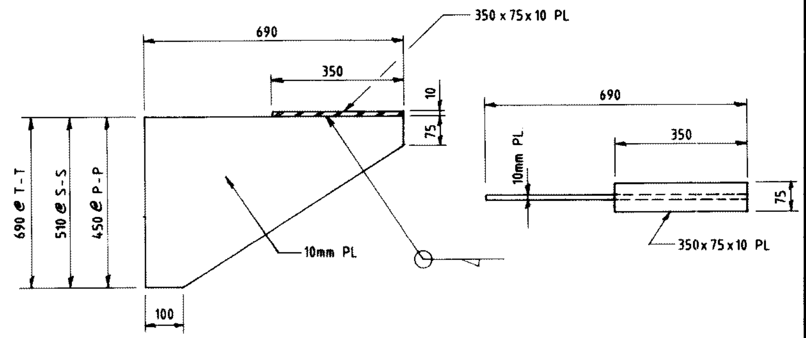
Sheet 2 of 8

FILE No.

FOR TENDER ONLY



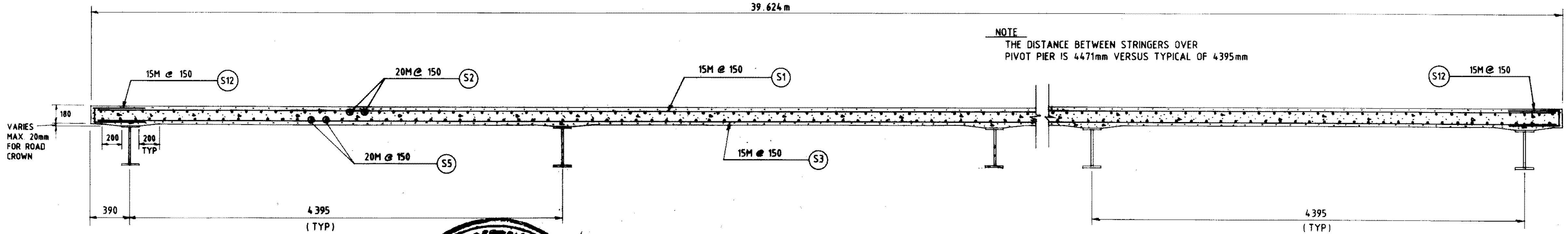
PLAN OF DECK STEEL
(SWING SPAN) SCALE 1 : 100



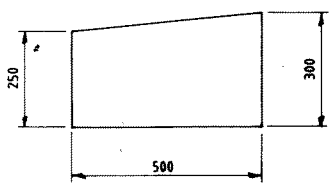
ELEVATION
SCALE 1 : 10

PLAN
SCALE 1 : 10

DETAILS OF GUSSET PLATE RIGHT



DECK REINFORCEMENT LONGITUDINAL SECTION
(SWING SPAN) SCALE 1 : 25
3 / 3 | 3

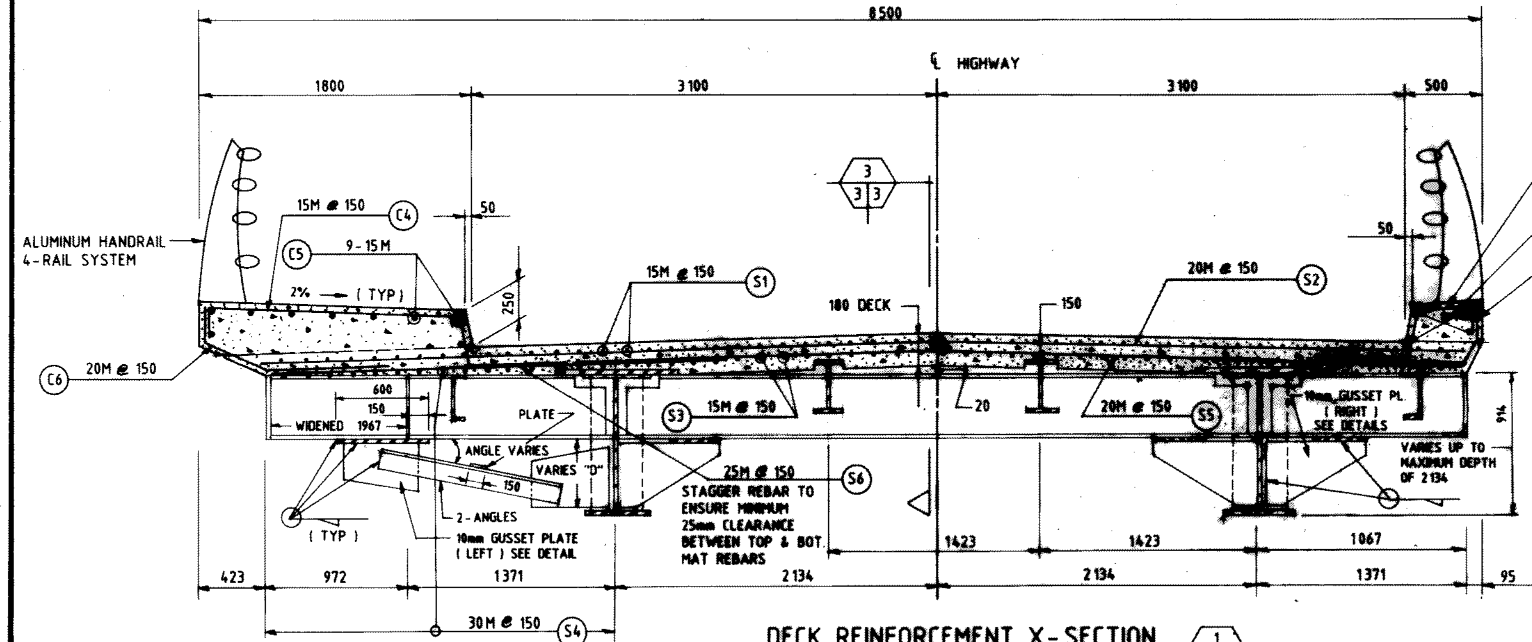


DETAIL OF GUSSET PLATE LEFT
SCALE 1 : 10

REGISTERED PROFESSIONAL ENGINEER
GARFIELD AU
SIGNATURE
April 26/91
DATE
PROVINCE OF NEWFOUNDLAND

DOMINION BRIDGE STEEL MEMBER DESIGNATION	EXISTING MEMBER SIZES (IMPERIAL)	QUANTITIES	REPLACE EXISTING MEMBER WITH FOLLOWING METRIC SIZES	WEIGHT PER MEMBER	TOTAL WT. FOR EACH MEMBER
15A	4x4x5/16∠ x 18'-11"	4	100x100x8∠ x 5.766M	69.8 Kg	279.2 Kg
15B	4x4x5/16∠ x 18'-11"	2	100x100x8∠ x 5.766M	69.8 Kg	139.6 Kg
15C	5x3 1/2x3/8∠ x 18'-10"	2	125x90x10∠ x 5.740M	92.4 Kg	184.8 Kg
15H	6x4x3/8∠ x 7'-11 1/16"	4	150x100x10∠ x 2.415M	45.4 Kg	181.6 Kg
15D	4x4x5/16∠ x 9'-3"	8	100x100x8∠ x 2.819M	34.1 Kg	272.8 Kg
15F	4x4x5/16∠ x 9'-3"	4	100x100x8∠ x 2.819M	34.1 Kg	136.4 kg
15G	5x3 1/2x3/8∠ x 9'-1 7/8"	4	125x90x10∠ x 2.791M	44.9 Kg	179.6 Kg
15K	3x2 1/2x5/16∠ x 6'-6"	8	90x65x8∠ x 1.981M	18.3 Kg	146.4 Kg
15M	3x2 1/2x5/16∠ x 7'-3 7/8"	8	90x65x8∠ x 2.232M	20.6 Kg	164.8 Kg
15N	3x2 1/2x5/16∠ x 12'-5 1/2"	4	90x65x8∠ x 3.797M	35.0 Kg	140.0 Kg
15P	3x2 1/2x5/16∠ x 12'-5 1/2"	4	90x65x8∠ x 3.797M	35.0 Kg	140.0 Kg
ga	6x3/8plx 1'-11"	4	150x10plx584	6.9 Kg	27.6 Kg
gb	7x3/8plx 2'-5"	2	178x10plx737	10.3 Kg	20.6 Kg
gc	9x3/8plx3'-1/2"	2	229x10plx927	16.7 Kg	33.4 Kg
gd	9 3/4x5/16plx 2'-1/4"	2	248x8plx616	9.6 Kg	19.2 Kg
gf	8 1/2x5/16plx 2'-5 1/4"	2	216x8plx743	10.1 Kg	20.2 Kg
15Tr	4x4x3/8∠ x 7'-9 3/8"	1	100x100x10∠ x 2.371M	35.3 Kg	35.3 Kg
15Tl	4x4x3/8∠ x 7'-9 3/8"	1	100x100x10∠ x 2.371M	35.3 Kg	35.3 Kg

EXISTING HORIZONTAL & VERTICAL
CROSS - BRACING TO BE REPLACED



NOTES
- DECK DRAINS OMITTED FOR CLARITY.
- ADDITIONAL HORIZONTAL CROSS - BRACING OMITTED FOR CLARITY.
- BOLTED CONNECTIONS ARE REQUIRED FOR REPLACEMENT OF EXISTING CROSS - BRACINGS (ANGLES).
- WELD CONNECTIONS FOR THE NEW STRUT SYSTEM & ADDITIONAL BRACINGS SHALL BE CONTINUOUS FILLET WELD (ALL AROUND) UNLESS OTHERWISE INDICATED.

DECK REINFORCEMENT X - SECTION
(SWING SPAN) SCALE 1 : 25
1 / 2 | 3

NEW STRUT MEMBERS	ANGLE (DEGREES)	D (DEPTH mm)	QUANTITIES	WEIGHT PER MEMBER	TOTAL WT. OF EACH MEMBER
125x125x16∠ x 1.50M @ P-P	11	470	4	44.1 Kg	176.4 Kg
125x125x16∠ x 1.50M @ S-S	11	470	4	44.1 Kg	176.4 Kg
100x100x13∠ x 1.57M @ T-T	20	700	4	30.0 Kg	120.0 Kg
100x100x10∠ x 1.71M @ M-M	30	1030	4	25.5 Kg	102.0 Kg
75x75x10∠ x 1.94M @ N-N	40	1420	4	21.3 Kg	85.2 Kg
200x150x10pl @ P-P & S-S	N/A	N/A	4	2.4 Kg	9.6 Kg
150x150x10pl @ T-T & M-M	N/A	N/A	4	1.8 Kg	7.2 Kg
140x150x10pl @ N-N	N/A	N/A	2	1.7 Kg	3.4 Kg
660x150x13pl	N/A	N/A	10	9.2 Kg	92.0 Kg
500x300x10pl	N/A	N/A	20	11.8 Kg	236.0 Kg
150x150x13∠ x 4.15M @ P-P & S-S *	N/A	N/A	8	121.6 Kg	972.8 Kg
125x125x10∠ x 4.15M @ T-T *	N/A	N/A	4	78.0 Kg	312.0 Kg
250x200x10pl @ P-P & S-S *	N/A	N/A	8	3.9 Kg	31.2 Kg
200x200x10pl @ T-T *	N/A	N/A	4	3.1 Kg	12.4 Kg
690x450x10pl @ P-P **	N/A	N/A	2	24.4 Kg	48.8 Kg
690x510x10pl @ S-S **	N/A	N/A	2	27.6 Kg	55.2 Kg
690x690x10pl @ T-T **	N/A	N/A	2	37.4 Kg	74.8 Kg
350x75x10pl @ P-P,S-S,T-T **	N/A	N/A	6	2.1 Kg	12.6 Kg

* ADDITIONAL HORIZONTAL CROSS - BRACINGS
** ADDITIONAL BRACING, GUSSET PLATE (RIGHT)

NEW STRUT MEMBERS & ADDITIONAL BRACINGS

—— NOTES ——

DO NOT SCALE DRAWING

9. ALL STRUCTURAL STEEL AND MEMBERS SHALL BE 300W IN ACCORDANCE WITH CSA G40.21.

10. ALL WELDING SHALL CONFORM TO CSA W59 AND W47-1 OF LATEST EDITION.

11. ALL SHOP DRAWINGS SHALL BE SUBMITTED TO THE ENGINEER FOR APPROVAL AND SHALL BEAR THE SEAL AND SIGNATURE OF A PROFESSIONAL ENGINEER WHO IS A MEMBER OF, OR IS LICENSED BY THE ASSOCIATION OF PROFESSIONAL ENGINEERS AND GEOSCIENTISTS OF NEWFOUNDLAND.

12. UNLESS OTHERWISE NOTED, MINIMUM FILLET WELD SIZES SHALL BE THE FOLLOWING:

MAXIMUM THICKNESS OF THICKER PART JOINED (mm)	MINIMUM SIZE OF FILLET WELD (mm)
1-12 inclusive	5
over 12 - 20	6
over 20 - 40	8
over 40 - 60	10
over 60 - 80	12

13. BOLTED CONNECTIONS SHALL BE GALVANIZED BOLTS C/W NUTS AND WASHERS IN ACCORDANCE WITH ASTM STANDARD A325 OR A325M.

14. BOLT HOLES MAY BE PUNCHED, SUB-PUNCHED OR SUB-DRILLED AND REAMED OR DRILLED.

NO.	BY	DESCRIPTION	DATE
		CONT ------ SHEET 4	

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

TITLE:
DECK REINFORCING DETAILS (SWING SPAN)

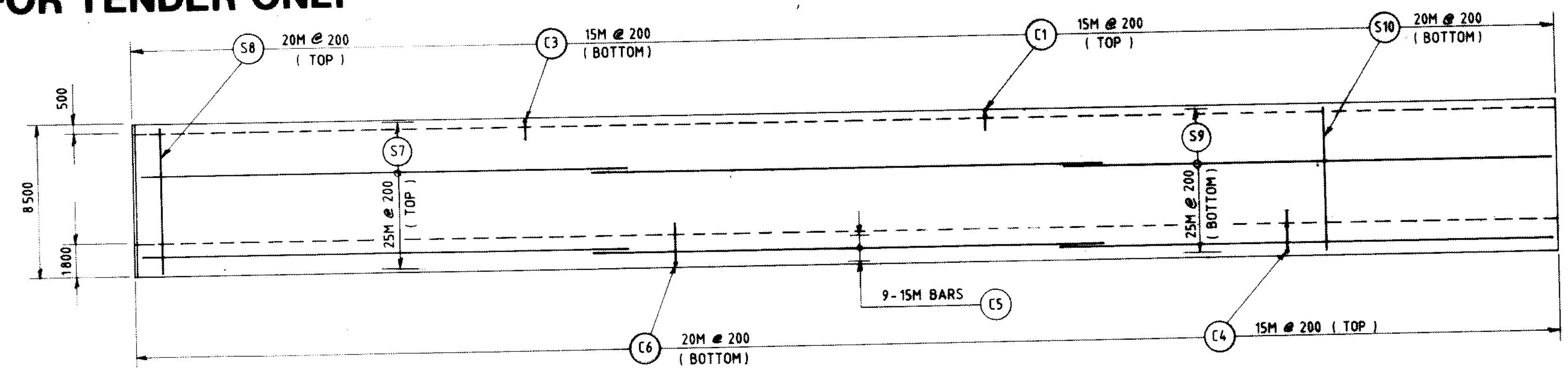
SCALE: AS SHOWN	DATE: 91/01/31	DRAWN BY: H. Jonas
DWG. CHK'D BY: G. Au	DESIGNED BY: G. Au	DES. CHK'D BY: BC
APPROVED BY: P.D.L.	DWG. NO.	FILE NO.

PROJECT NO.
7-91 PSR

Sheet 3 of 8

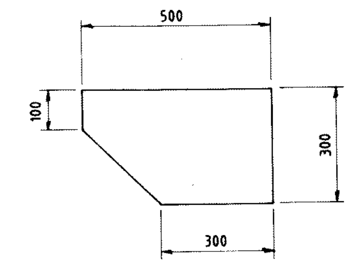
FILE No. ______

FOR TENDER ONLY

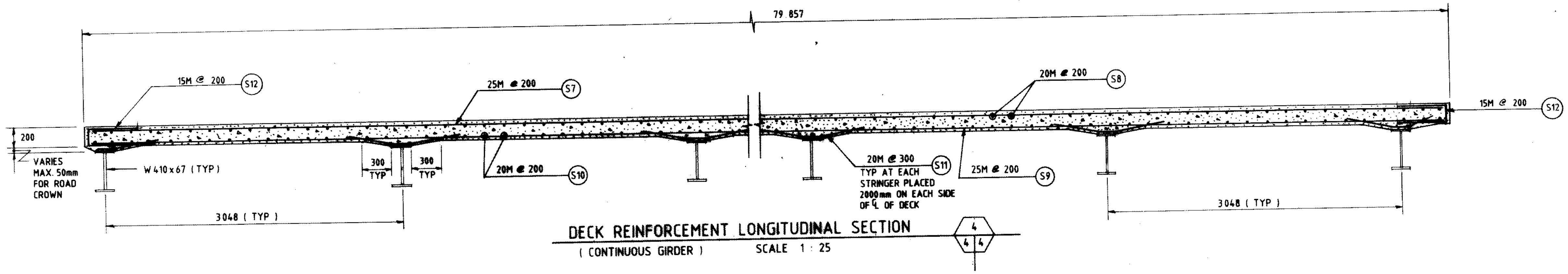


PLAN OF DECK STEEL
(CONTINUOUS GIRDER) SCALE 1 : 200

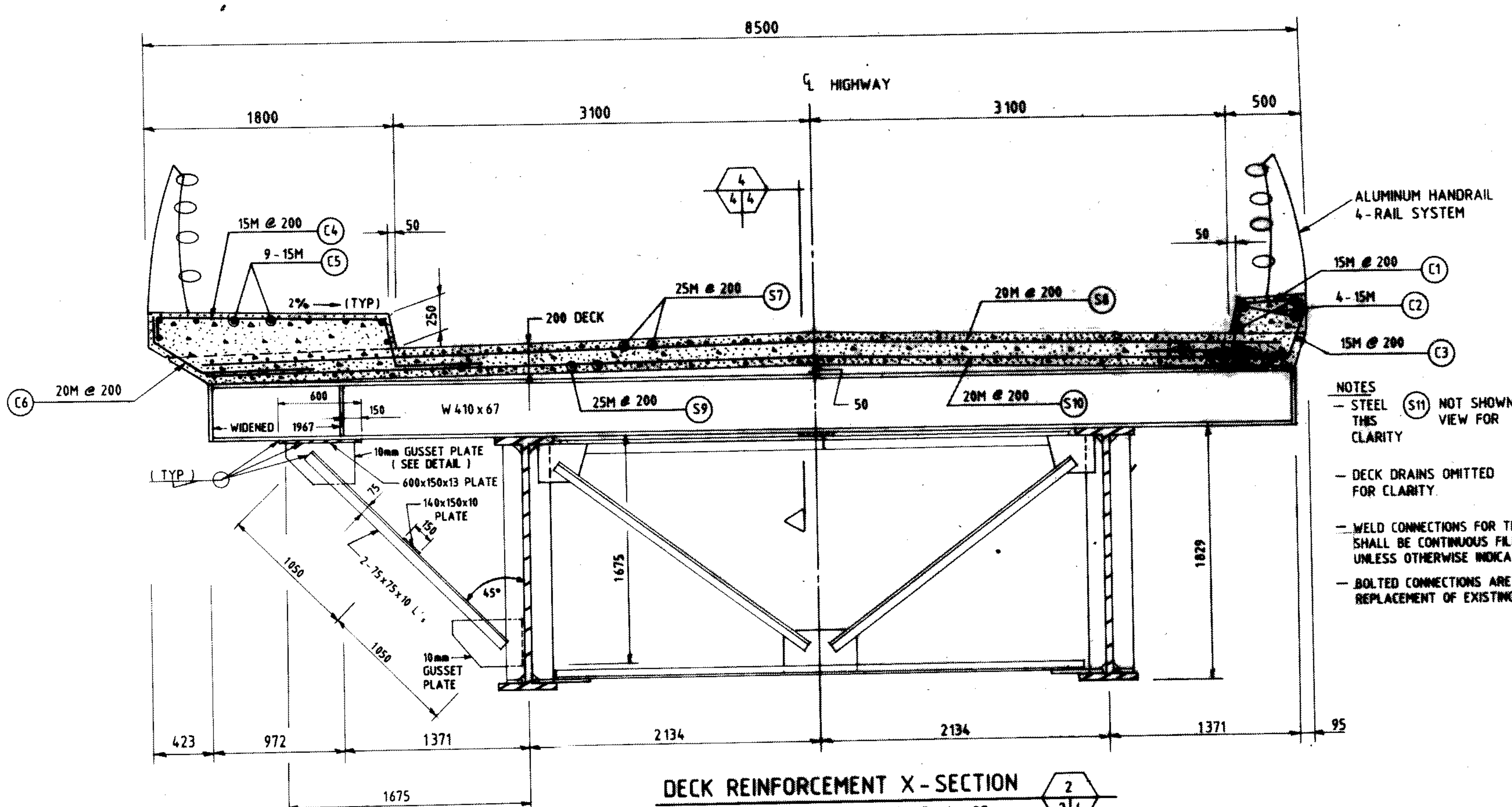
NOTE
– STEEL VIEW S11 NOT SHOWN THIS
– STEEL VIEW C2 NOT SHOWN THIS



DETAIL OF GUSSET PLATE
SCALE 1 : 10



DECK REINFORCEMENT LONGITUDINAL SECTION
(CONTINUOUS GIRDER) SCALE 1 : 25



NOTES
– STEEL S11 NOT SHOWN THIS VIEW FOR CLARITY
– DECK DRAINS OMITTED FOR CLARITY
– WELD CONNECTIONS FOR THE NEW STRUT SYSTEM SHALL BE CONTINUOUS FILLET WELD (ALL AROUND) UNLESS OTHERWISE INDICATED
– BOLTED CONNECTIONS ARE REQUIRED FOR REPLACEMENT OF EXISTING CROSS - BRACINGS (ANGLES)

DECK REINFORCEMENT X-SECTION
(CONTINUOUS GIRDER) SCALE 1 : 25

DOMINION BRIDGE STEEL MEMBER DESIGNATION	EXISTING MEMBER SIZES (IMPERIAL)	QUANTITIES	REPLACE EXISTING MEMBER WITH FOLLOWING METRIC SIZES	WEIGHT PER MEMBER	TOTAL WT. FOR EACH MEMBER
6A	4x4x5/16∠ x 22'–6 5/8"	13	100x100x8 ∠x 6.925M	83.8 Kg	1089.4 Kg
6B	4x4x5/16∠ x 10'–10 11/16"	26	100x100x8∠ x 3.319M	40.2 Kg	1045.2 Kg
6C	3x2 1/2x5/16∠ x 7'–3 3/16"	8	90x65x8 ∠x 2.215M	20.4 Kg	163.2 kg
6D	3x2 1/2x5/16∠ x 6'–8 7/8"	8	90x65x8 ∠x 2.054M	19.0 Kg	152.0 Kg
6F	3x2 1/2x5/16∠ x 8'–0"	40	90x65x8∠ x 2.439M	22.5 Kg	900.0 Kg
6G	3x2 1/2x5/16∠ x 12'–9"	20	90x65x8∠ x 3.887M	35.9 Kg	718.0 Kg
9a	10 3/8"x3/8pl x 1'–9 1/2"	10	264x10pl x646	11.3 Kg	113.0 Kg

EXISTING HORIZONTAL & VERTICAL CROSS - BRACING TO BE REPLACED

NEW STRUT MEMBERS	QUANTITIES	WEIGHT PER MEMBER	TOTAL WT. OF EACH MEMBER
600x150x13pl	27	9.2 Kg	248.4 Kg
75x75x10∠x 2.1M	54	23.1 Kg	1247.4 Kg
140x150x10pl	27	1.7 Kg	45.9 Kg
500x300x10pl	54	11.8 Kg	637.2 Kg

NEW STRUT MEMBERS

REGISTERED PROFESSIONAL ENGINEER
GARFIELD AU
APRIL 26/91
PROVINCE OF NEWFOUNDLAND

— NOTES —

DO NOT SCALE DRAWING

15. ALL FINISHED STEEL SUPERSTRUCTURE (NEW & EXISTING), EXISTING BEARINGS OR AS DIRECTED BY THE ENGINEER SHALL BE PAINTED IN ACCORDANCE WITH SECTION 921 "BLAST CLEANING AND PAINTING OF STRUCTURAL STEEL" OF THE GENERAL SPECIFICATION BOOK OF DEPARTMENT OF WORKS, SERVICES AND TRANSPORTATION.

16. PREVIOUS STEEL CONTRACT DRAWINGS BY DOMINION BRIDGE CO. LTD. DATED 1958 MAY BE VIEWED AT ST. JOHN'S HIGHWAY DESIGN DIVISION.

NO.	BY	DESCRIPTION	DATE

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

TITLE:
DECK REINFORCING DETAILS (CONTINUOUS GIRDER)

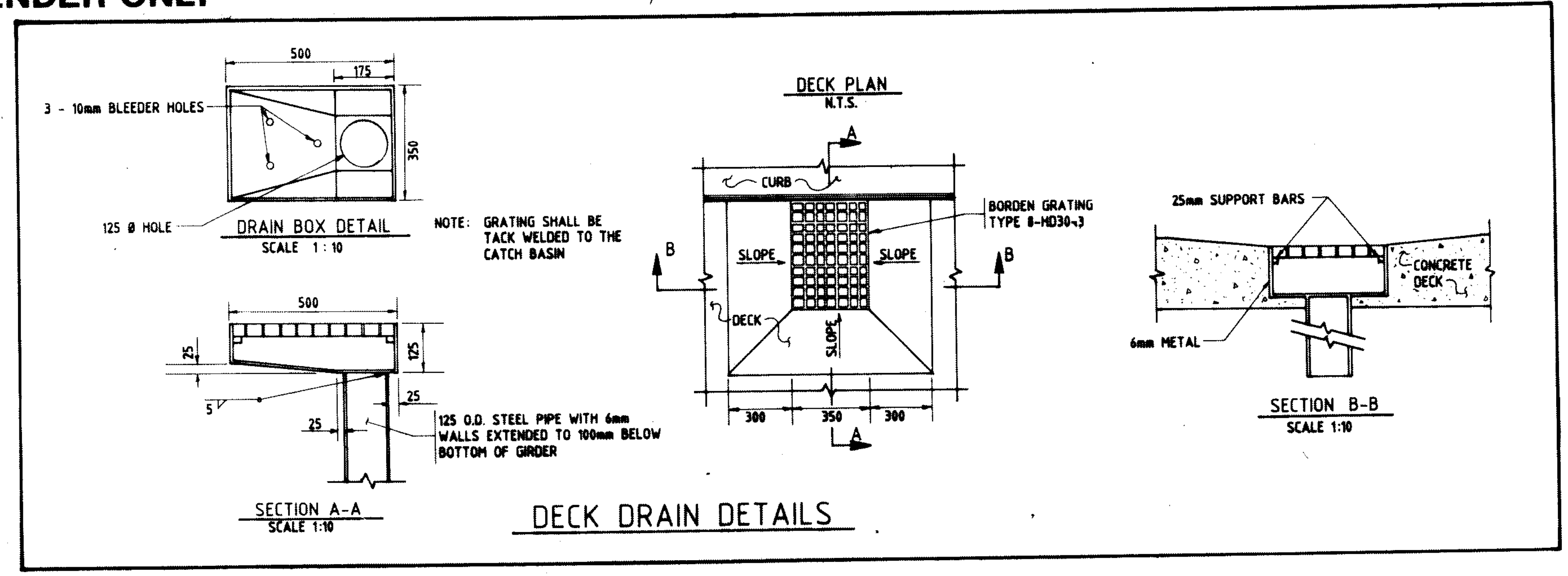
SCALE: AS SHOWN	DATE: 91/01/31	DRAWN BY: H. Hones
DWG. CHK'D. BY: G. Au	DESIGNED BY: G. Au	DES. CHK'D BY:
APPROVED BY: P.D.L.	DWG. NO.	FILE NO.

PROJECT NO. 7-91PSR

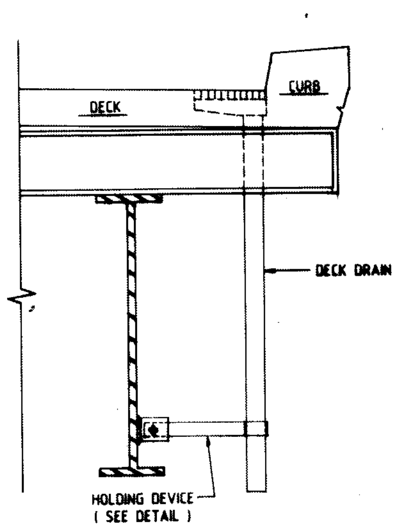
Sheet 4 of 8

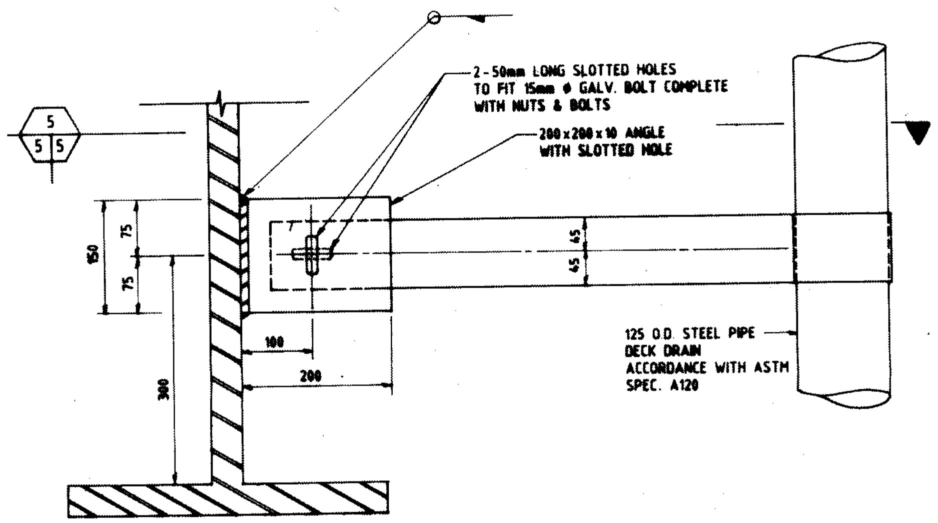
FILE No.____

FOR TENDER ONLY

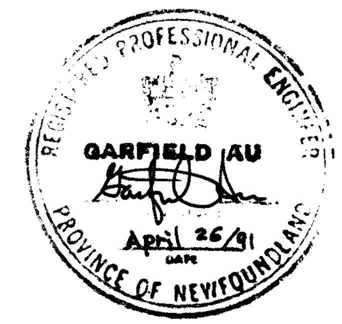


DECK DRAIN DETAILS

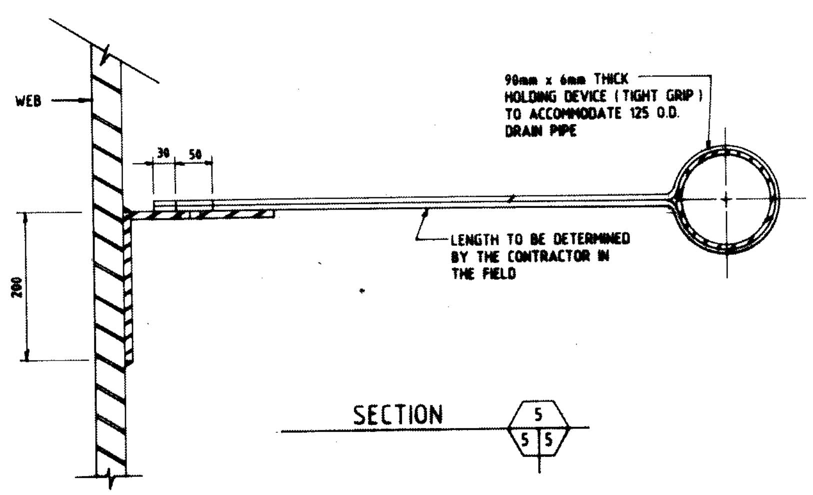




HOLDING DEVICE DETAIL



NOTE
ALL DECK DRAIN SYSTEM SHALL BE HOT DIP GALVINIZED TO CSA STANDARD G164



SECTION 5 / 5 5

DRAIN PIPE BRACING DETAILS
SCALE N.T.S.

FILE No.______

— NOTES —
DO NOT SCALE DRAWING

NO.	BY	DESCRIPTION	DATE

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

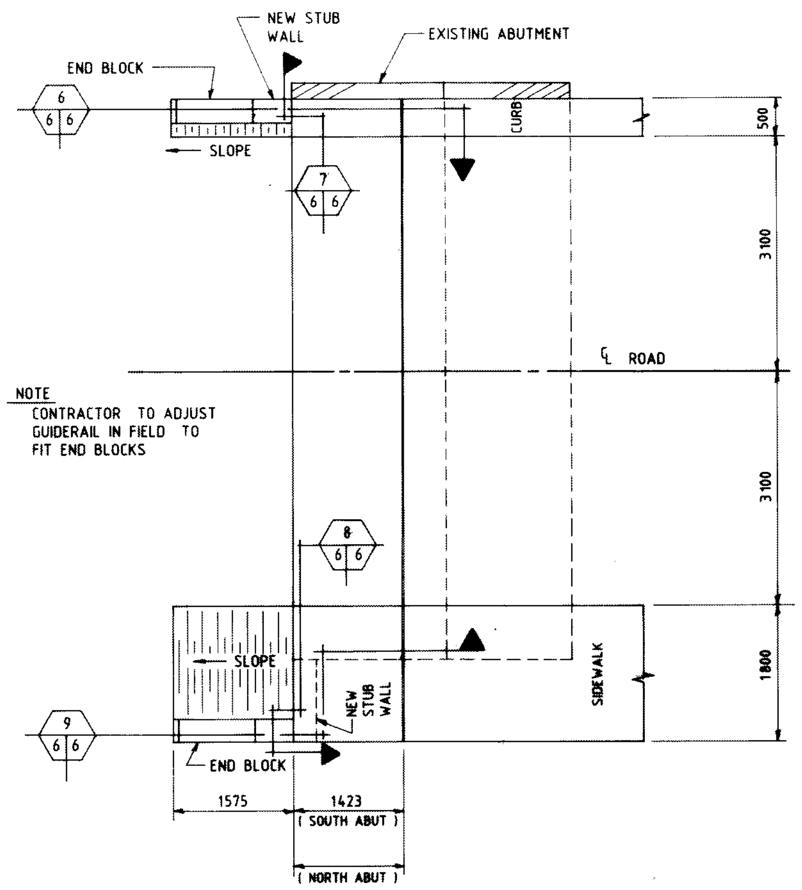
TITLE:
DECK DRAINS
&
DRAIN PIPE BRACING DETAILS

SCALE: AS SHOWN	DATE: 91/03/19	DRAWN BY:
DWG. CHK'D. BY: G.Au	DESIGNED BY: G.Au	DSG. CHK'D BY:
APPROVED BY: P.O.L.	DWG. NO.	FILE NO.

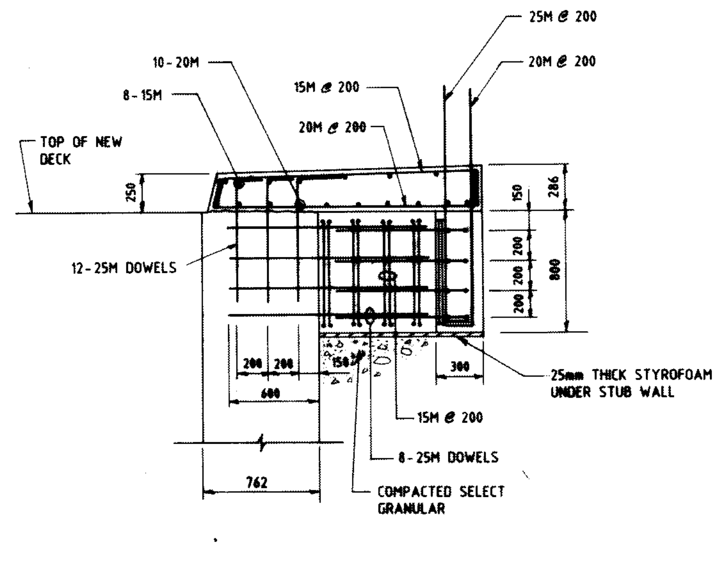
PROJECT NO.
7-91PSR

Sheet 5 of 8

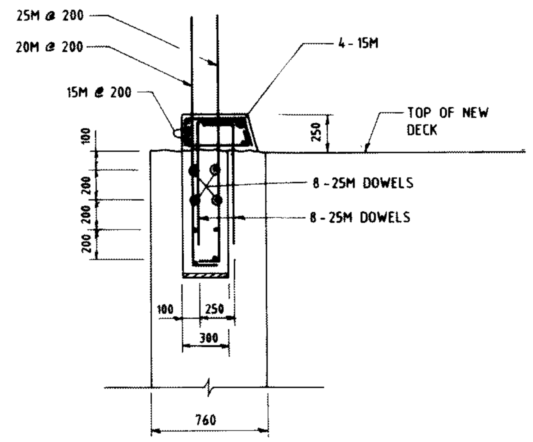
FOR TENDER ONLY



SIDEWALK & CURB DETAIL PLAN
(NORTH & SOUTH ABUTMENT) SCALE N.T.S.



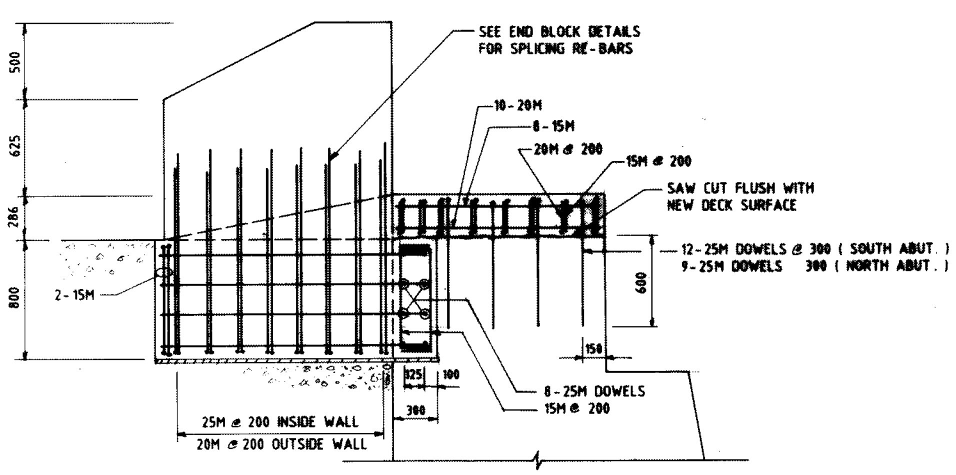
SECTION
SCALE N.T.S.



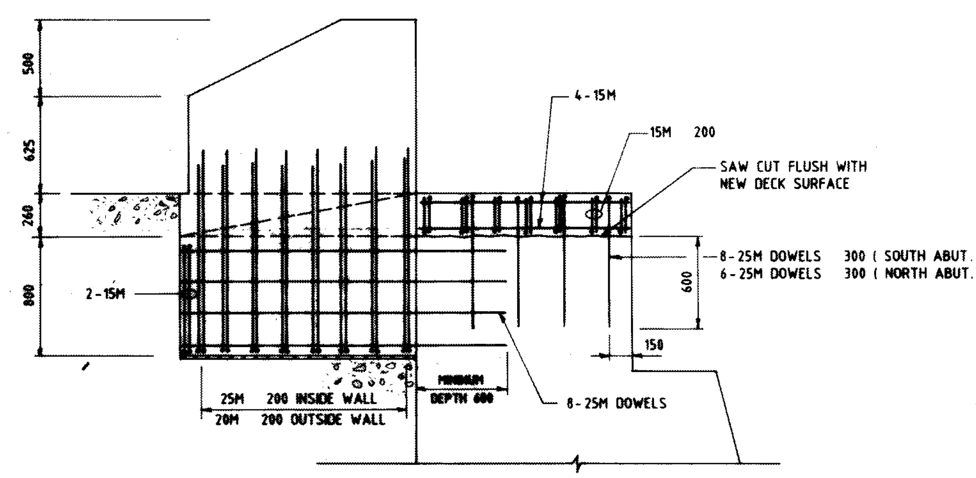
SECTION
SCALE N.T.S.

NOTES
- TOP & SIDE OF NEW SIDEWALK & CURB ON ABUTMENT SHALL BE FLUSH WITH NEW SIDEWALK & CURB ON DECK
- HOLES FOR DOWELS SHALL BE DRILLED WITH COREBIT FOR PRECISION PURPOSES

GARFIELD AU
April 26/91



SECTION
SCALE N.T.S.



SECTION
SCALE N.T.S.

NOTES
DO NOT SCALE DRAWING

NO.	BY	DESCRIPTION	DATE

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

TITLE:
SIDEWALK & CURB DETAILS AT NORTH & SOUTH ABUTMENTS

SCALE: AS SHOWN	DATE: 91/04/24	DRAWN BY:
DWG. CHK'D. BY: G.A.	DESIGNED BY: G.A.	DES. CHK'D BY:
APPROVED BY: P.D.L.	DWG. NO.	FILE NO.

PROJECT NO.
7-91PSR

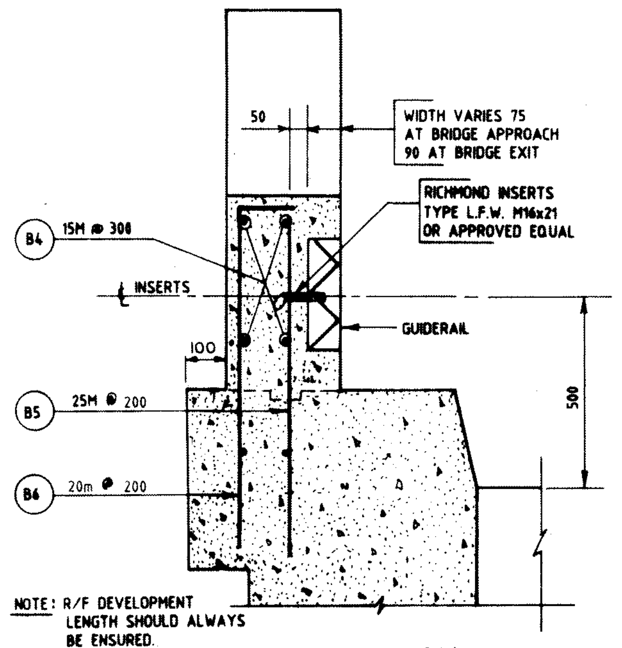
Sheet 6 of 8

FILE No.

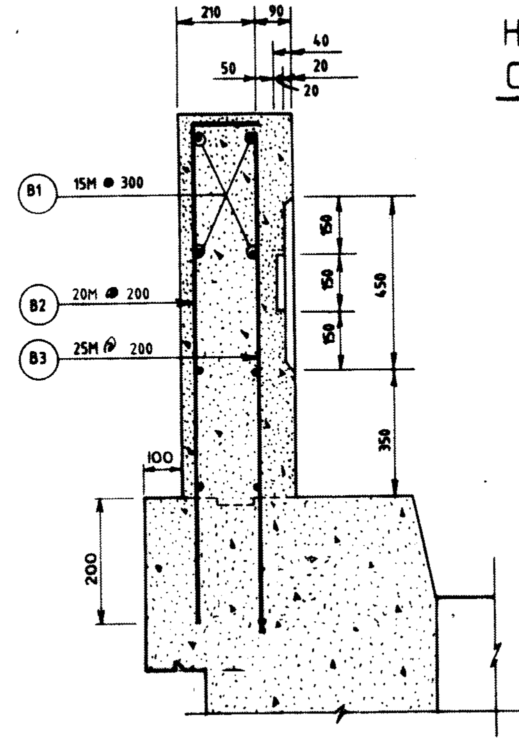
FOR TENDER ONLY

HANDRAIL & END BLOCK GUIDERAIL CONNECTION DETAILS

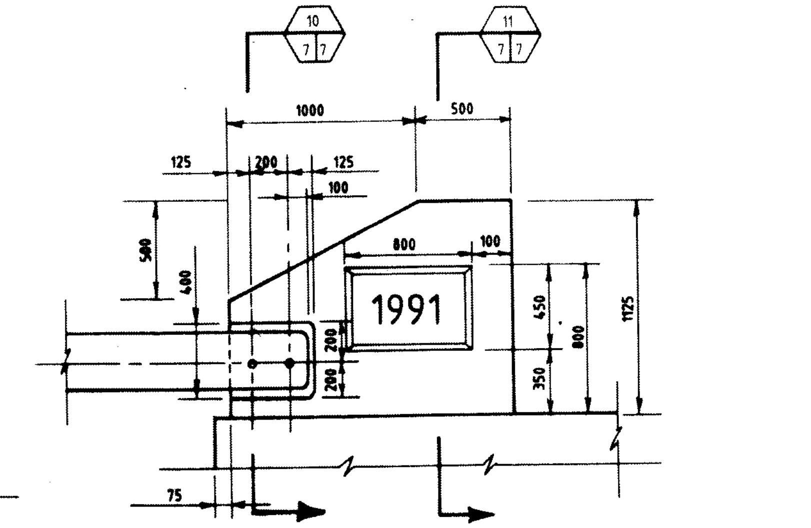
NOTE: 1. A RECESS IS REQUIRED IN ALL END BLOCKS WITHOUT NUMERALS FOR REPAIR PROJECTS.
2. THE DEPARTMENT SHALL SUPPLY TWO (2) 800 x 450 x 19 PLYWOOD PANELS, C/W NUMERALS ATTACHED, AT THE NEAREST DISTRICT OFFICE. FOR NEW PROJECTS ONLY



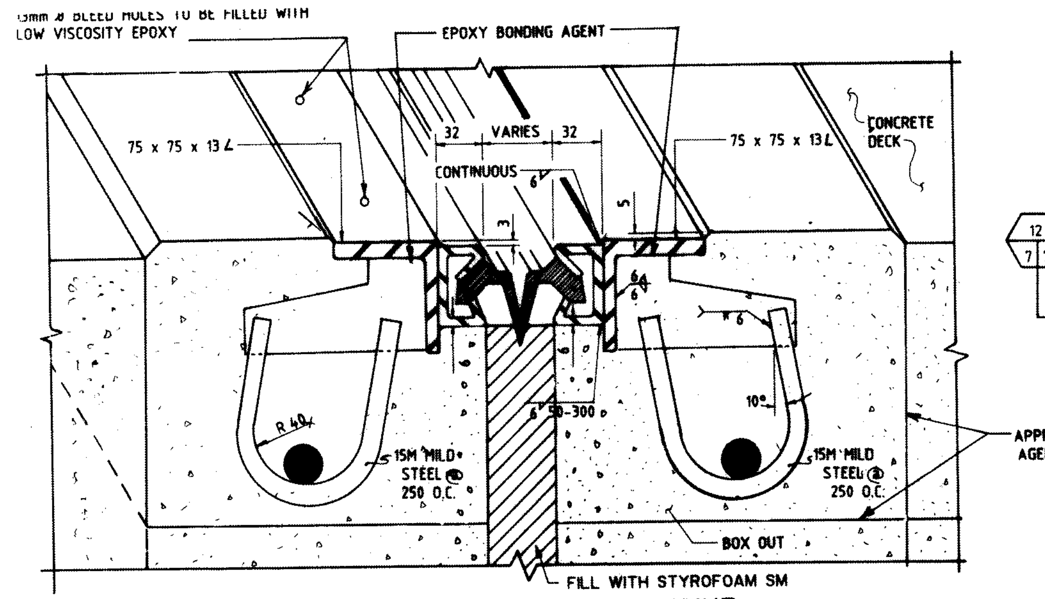
SECTION 10/7/7
SCALE 1:10



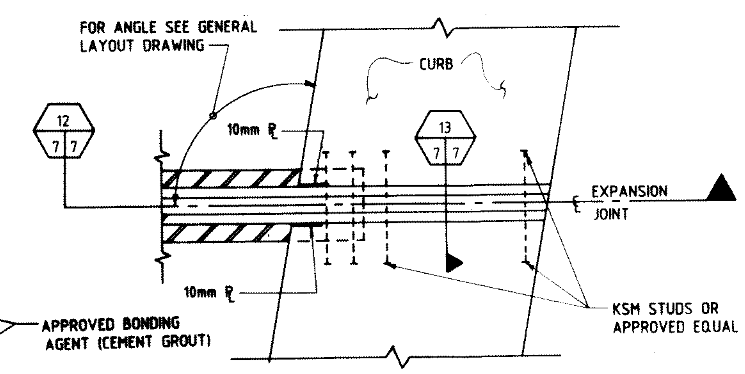
SECTION 11/7/7
SCALE 1:10



ELEVATION OF GUIDERAIL CONNECTION
SCALE 1:20

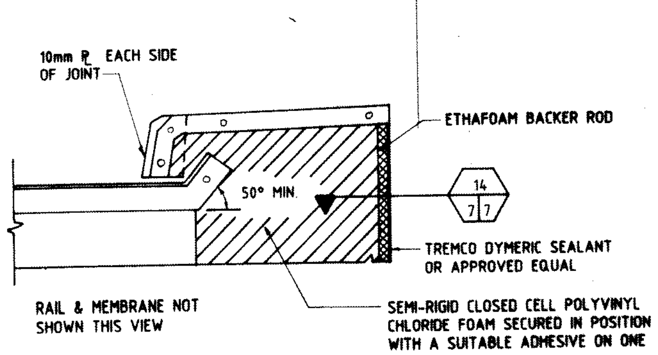


TYPICAL SECTION FOR EXPANSION JOINT
N.T.S.

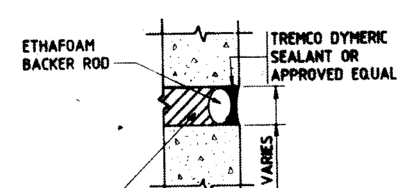


EXPANSION JOINT PLAN
N.T.S.

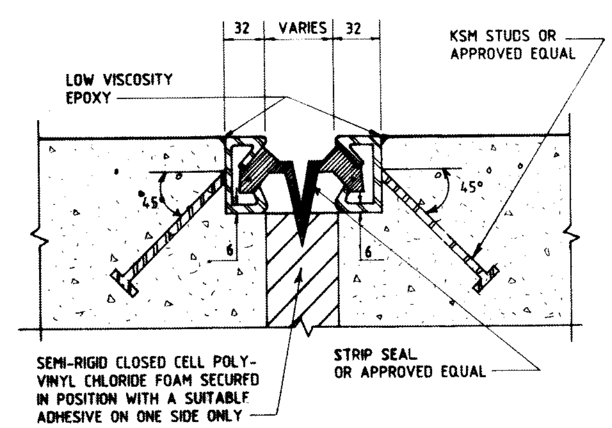
APPROVED LIST
Z-TECH ACME STELENCO



SECTION 12/7/7
N.T.S.



SECTION 14/7/7
N.T.S.



SECTION 13/7/7
N.T.S.

EXPANSION JOINT DETAILS

FILE No. ____

NOTES

DO NOT SCALE DRAWING

NO.	BY	DESCRIPTION	DATE

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT:
REHABILITATION OF MARYSTOWN CANNING BRIDGE

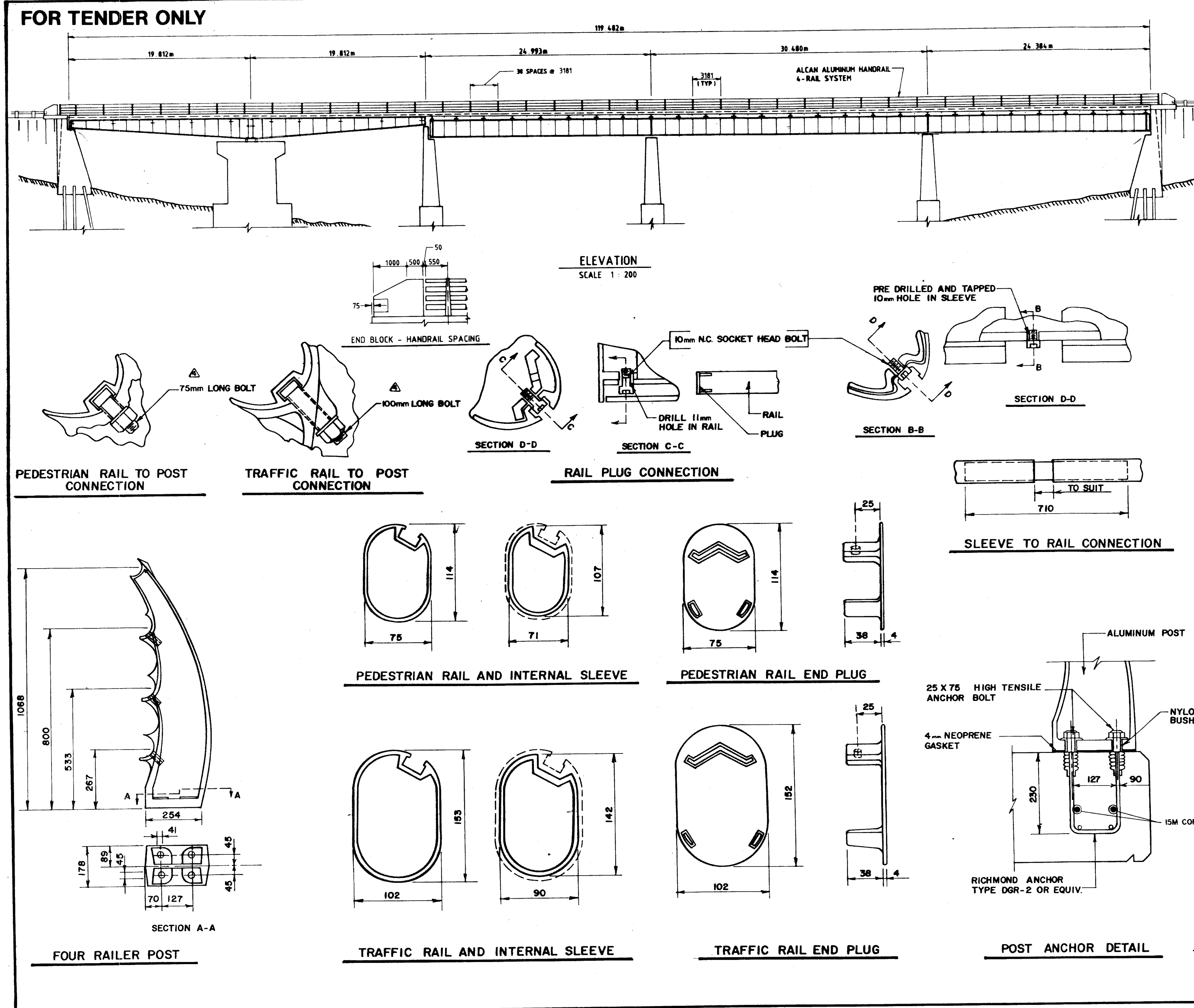
TITLE:
EXPANSION JOINT, HANDRAIL & END BLOCK DETAILS

SCALE: AS SHOWN	DATE: 91/04/24	DRAWN BY: D. [signature]
DWG. CHK'D BY:	DESIGNED BY:	DES. CHK'D BY:
APPROVED BY:	DWG. NO.	FILE NO.

PROJECT NO.
7-91 PSR

Sheet 7 of 8

FOR TENDER ONLY



FILE No.

NOTES

1. ALL POST SPACING MUST BE EQUAL. MAXIMUM POST SPACING TO BE 3658
2. RAIL TO RAIL JOINTS MIN. 600 FROM NEAREST POST.
3. SLEEVES FOR RAIL TO RAIL JOINTS MUST BE 710 mm IN LENGTH
4. A GAP OF 25 mm MUST BE LEFT BETWEEN RAIL SECTIONS.
5. A RAIL JOINT MUST BE PLACED IN SPACING OVER EXPANSION JOINT, UNLESS NOTED.
6. BOLTS ATTACHING THE PEDESTRIAN RAIL TO THE POST (16 mm x 45 mm) SHALL BE TORQUED BY 1/2 ROTATION FROM SNUG-TIGHT CONDITION, UNLESS NOTED.
7. BOLTS ATTACHING THE TRAFFIC RAILS TO POST (16 mm x 75 mm) SHALL BE TORQUED BY 1/2 ROTATION FROM SNUG-TIGHT CONDITION, UNLESS NOTED.
8. BOLTS ATTACHING POSTS TO ANCHORS (25 mm x 76 mm) SHALL BE TORQUED BY 1/3 ROTATION FROM SNUG-TIGHT CONDITION, UNLESS NOTED.
9. NUMBER OF SPLICES TO BE KEPT AT AN ABSOLUTE MINIMUM WITH THE LONGEST OPTIMUM RAIL LENGTHS EMPLOYED.
10. RAIL TO BE CONTINUOUS OVER AT LEAST TWO POSTS, PROVIDED POST TO RAIL CONNECTION IS SECURE, ie. NOT HAND TIGHTENED ONLY.
11. NORMAL MINIMUM LENGTH OF RAIL SHALL BE 9.2m UNLESS SHORTER LENGTHS ARE REQUIRED TO MEET DESIGN CRITERIA.

NO.	BY	DESCRIPTION	DATE
4	B.I.	BOLT LENGTHS INCREASED	91/02/15
3	C.B.	BOLTS LENGTHENED & NOTE 11 ADDED	84-01-03
2	kap	NOTES 9 + 10 ADDED	85-03-27
1	CB	NOTES 6,7&8 ADDED	85-08-05

REVISIONS

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET LOCATION
C. DETAIL SHEET

GOVERNMENT OF NEWFOUNDLAND AND LABRADOR

DEPARTMENT OF
WORKS SERVICES AND TRANSPORTATION
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

PROJECT
REHABILITATION OF MARYSTOWN CANNING BRIDGE

TITLE:
HANDRAIL DETAILS

SCALE: N.T.S.	DATE: DEC. 1983	DRAWN BY:
DWG. CHK'D. BY.	DESIGNED BY:	DES. CHK'D BY:
APPROVED BY:	DWG. NO.	FILE NO.

PROJECT NO.
7-91 PSR

Sheet 8 of 8

CUSTOM SIGNS

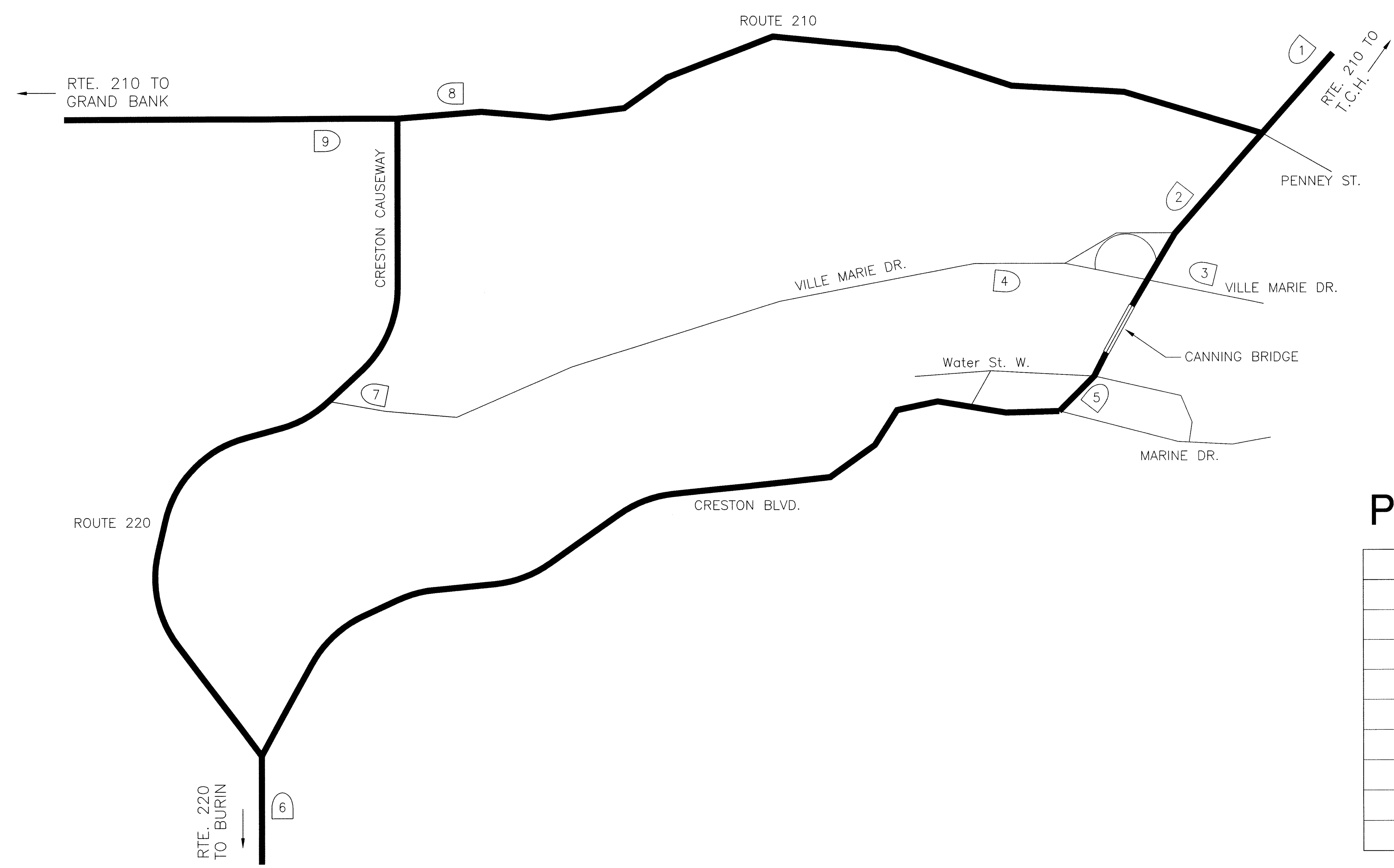
SIGN No.	SIGN LAYOUT	POST TYPE	QTY.
1	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY / ↑ LOCAL TRUCK TRAFFIC ONLY / SUGGESTED TRUCK DETOUR ROUTE →	D–2440/2440 6" & 5" TEXT	1
2	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY / → TRUCKS USE VILLE MARIE DRIVE	C–2440/1220 4" TEXT	1
3	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY / ↑ TRUCKS USE VILLE MARIE DRIVE	C–2440/1220 4" TEXT	1
4	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY / NO LEFT TURN	C–1830/1220 4" TEXT	1

CUSTOM SIGNS

SIGN No.	SIGN LAYOUT	POST TYPE	QTY.
5	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY	C–915/1220 4" TEXT	1
6	CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY / ↑ SUGGESTED TRUCK DETOUR ROUTE / LOCAL TRUCK TRAFFIC ONLY →	D–2440/2440 6" & 5" TEXT	1
7	TRUCK DETOUR ROUTE / ← Burin / Grand Bank →	C–1220/2440 6" TEXT	1
8	CANNING BRIDGE TRUCK DETOUR ROUTE ←	C–1220/2440 6" TEXT	1

CUSTOM SIGNS

SIGN No.	SIGN LAYOUT	POST TYPE	QTY.
9	NOTICE TO TRUCKERS / CANNING BRIDGE RESTRICTED TO PASSENGER VEHICLES ONLY	D–1830/2440 6" TEXT	1



POST REQUIREMENTS

POST TYPE	QTY.	POST TYPE	QTY.
C–915/1220	1		
C–1220/2440	2		
C–1830/1220	1		
C–2440/1220	2		
D–1830/2440	1		
D–2440/2440	2		

Newfoundland Labrador

TRANSPORTATION AND WORKS
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

1. EXACT SIGN LOCATIONS TO BE DETERMINED IN THE FIELD.
2. THE CUSTOM SIGNS NOTED ON THIS SIGN SHEET WILL BE SUPPLIED BY THE DEPARTMENT AND ARE TO BE INSTALLED BY THE CONTRACTOR.
3. THE CONTRACTOR SHALL BE RESPONSIBLE TO SUPPLY & INSTALL ALL TYPICAL CONSTRUCTION SIGNS AS SHOWN IN THE TRAFFIC CONTROL MANUAL, MAY 2010 EDITION.
4. FOR TYPICAL CONSTRUCTION SIGNAGE, REFER TO DIAGRAM 752–4, LANE CLOSED, LONG TERM WORK, AND 756–2, INTERSECTING ROADS IN WORK AREAS, AND SIGN ACCORDINGLY.
5. THE SPEED LIMIT THROUGH THE WORK AREA SHALL BE POSTED TO A MAXIMUM OF 30km/h.
6. A MINIMUM LANE WIDTH OF 3.0m SHALL BE MAINTAINED THROUGH THE WORK ZONE.

DO NOT SCALE DRAWINGS

No.	By	REVISION DESCRIPTION	Date

Project/Drawing Title

ROUTE 220
SIGN PLAN FOR ONE LANE CLOSURE ON CANNING BRIDGE
SIGN SHEET

TRAFFIC ENGINEERING DIVISION

Design Engineer

Designed By

Date

Des Chk'd By

Date

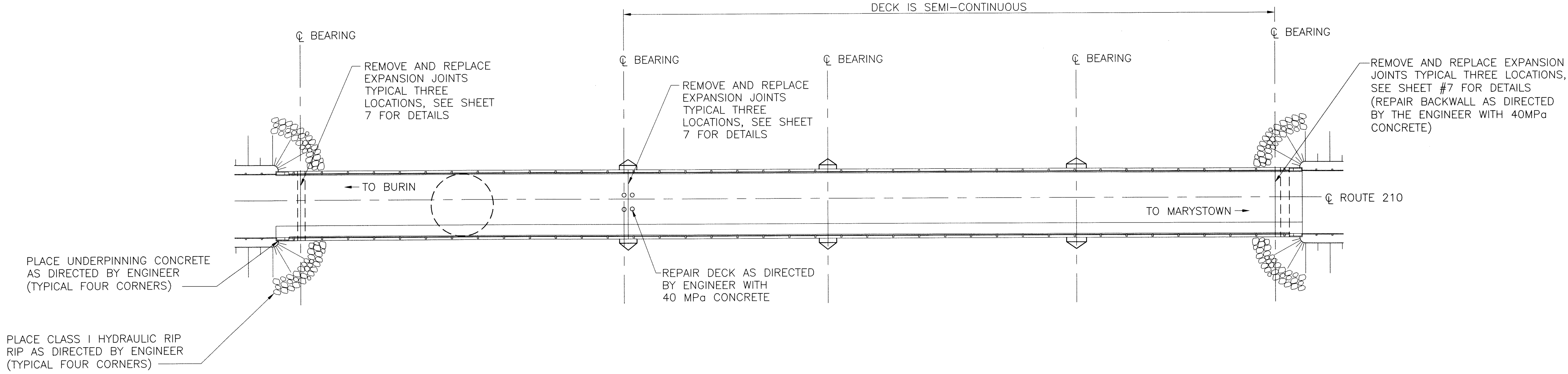
D. OSMOND

" FOR CONSTRUCTION "

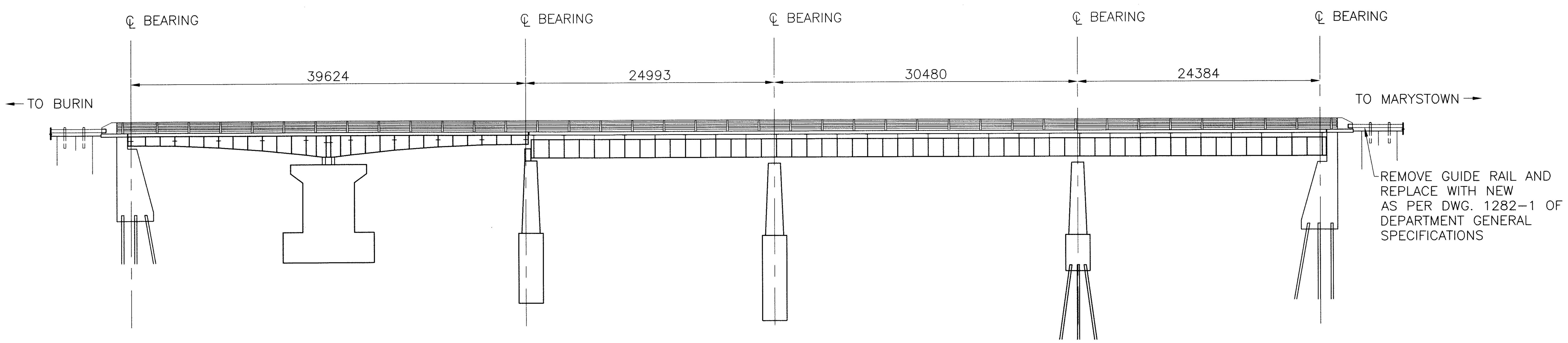
Project No. 35–10 PSR

FOR TENDER

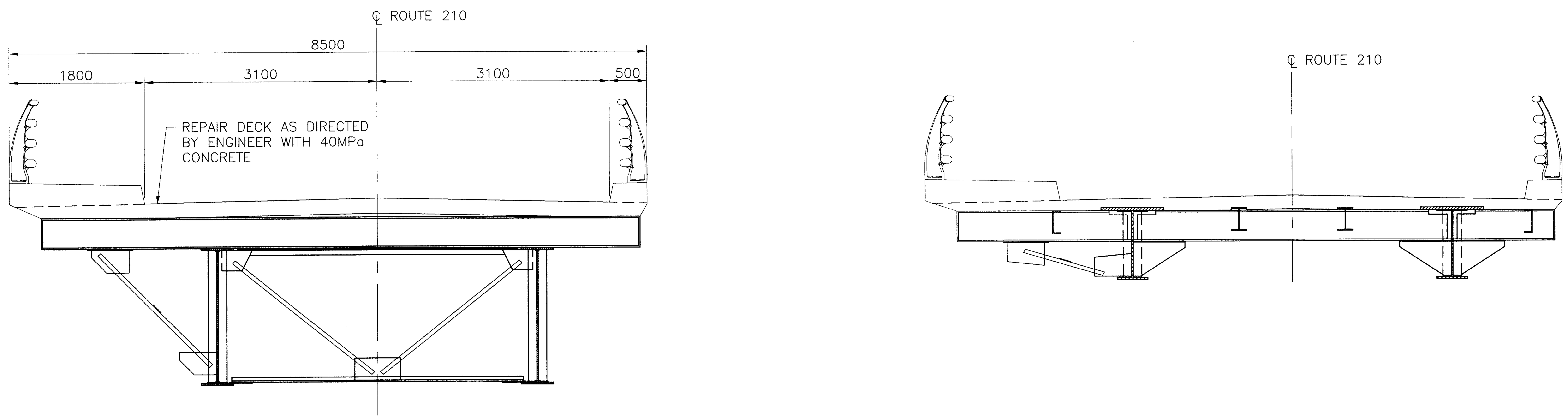
Sheet No. 4 of 8



EXISTING PLAN
SCALE 1 : 300



EXISTING ELEVATION
SCALE 1 : 300



EXISTING SECTIONS
SCALE 1 : 40

Site No. 2-003

Newfoundland Labrador

TRANSPORTATION AND WORKS
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

1. ALL WORK SHALL BE IN ACCORDANCE WITH CAN/CSA-S6-06 DESIGN OF HIGHWAY BRIDGES AND CAN/CSA-A23.1-M04 CONCRETE MATERIALS AND METHODS OF CONCRETE CONSTRUCTION.
2. ORIGINAL DESIGN LIVE LOAD HS20-S16-44.
3. DIMENSIONS SHOWN ARE TAKEN FROM ORIGINAL DRAWINGS. ACTUAL FIELD DIMENSIONS MAY NOT AGREE. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO VERIFY ALL DIMENSIONS.
4. THE CONTRACTOR SHALL TAKE CARE TO MAINTAIN STRUCTURAL INTERGRITY OF BEARINGS AT ALL TIMES. THE ENGINEER WILL DETERMINE WHETHER OR NOT GIRDERS HAVE TO BE PROPPED DURING CONCRETE REMOVAL.
5. A 30mm DEEP SAWCUT IS REQUIRED AROUND ALL DETERIORATED AREAS PRIOR TO CONCRETE REMOVAL.
6. CONCRETE SHALL BE 35MPa IN 7 DAYS AND 40MPa IN 28 DAYS.
7. ALL ACCUMULATED DEBRIS ON BEAM SEATS AND EXISTING FORMWORK REMAINING FORM PREVIOUS CONSTRUCTION AT BACKWALLS AND DIAPHRAGMS SHALL BE REMOVED.
8. NOT LESS THAN 25mm BEVELS SHALL BE PLACED AT ANGLES OF FORMS TO CHAMFER CORNERS.
9. SURFACE FINISH CLASS II ON ALL SURFACES EXCEPT EXCEPT DECK AND CURBS WHICH SHALL BE CLASS VI.
10.

BAR NUMBER	SPLICE
10	500
15	600
20	800
25	1200
30	1800
35	2400

A) CONCRETE CAST AGAINST EARTH	75mm
B) CONCRETE EXPOSED TO WATER AND WEATHER	50mm
C) DECK – TOP REINFORCING	65mm
D) DECK – BOTTOM REINFORCING	30mm
E) CURB REINFORCING	65mm

No.	By	REVISION DESCRIPTION	Date

FOR USE BY TRAFFIC ENGINEERING ONLY

Designed By	Date	Checked By	Date

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET WHERE TAKEN
C. SHEET WHERE DETAILED

DO NOT SCALE DRAWINGS

Project/Drawing Title

PART "C"
CANNING BRIDGE
EXISTING GENERAL ARRANGEMENT

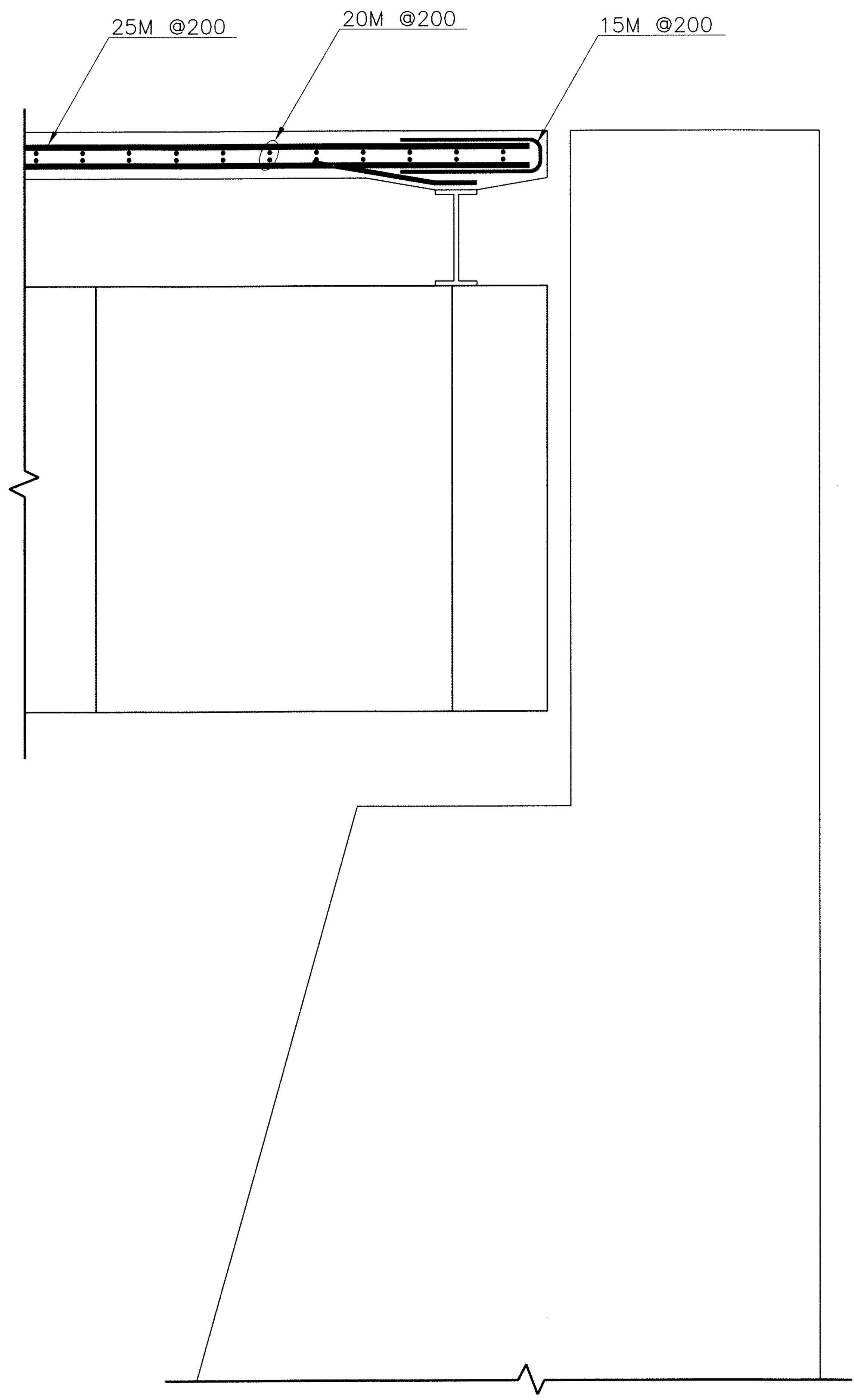
Drawn By: SN/ML
Dwg Chk'd By:
Designed By:
Des Chk'd By:
Approved By:
Date: 05-17-10

Design Engineer

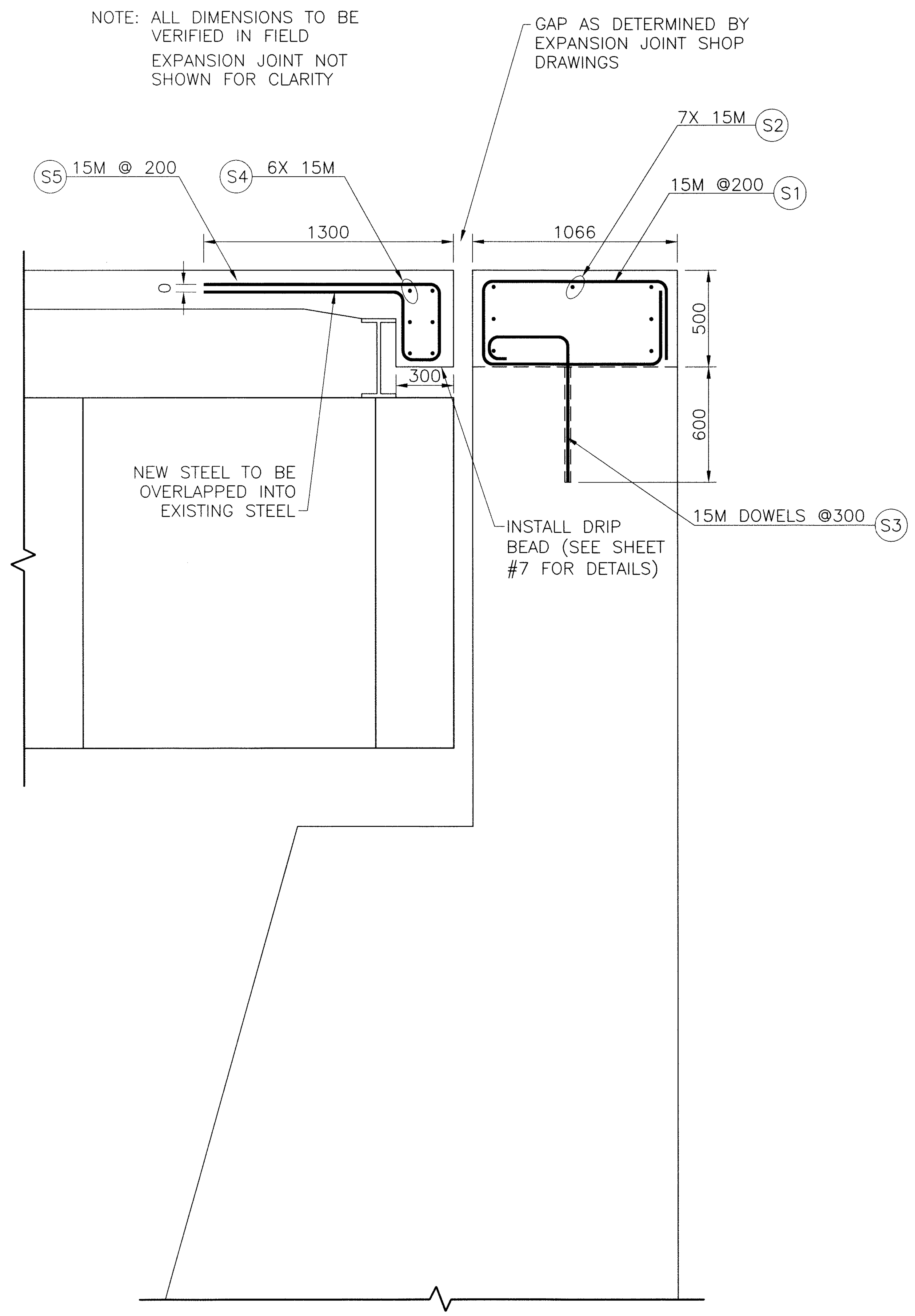
REGISTERED PROFESSIONAL ENGINEER
R. G. MATTHEWS
R.G. Matthews
07/08/10
PROVINCE OF NEWFOUNDLAND

" FOR CONSTRUCTION "

Project No. 35-10 PSR



EXISTING NORTH ABUTMENT
SCALE 1 : 20



PROPOSED NORTH ABUTMENT
SCALE 1 : 20

Site No. 2-003

Newfoundland Labrador

TRANSPORTATION AND WORKS
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

1. ALL WORK SHALL BE IN ACCORDANCE WITH CAN/CSA–S6–06 DESIGN OF HIGHWAY BRIDGES AND CAN/CSA–A23.1–M04 CONCRETE MATERIALS AND METHODS OF CONCRETE CONSTRUCTION.
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7. ALL ACCUMULATED DEBRIS ON BEAM SEATS AND EXISTING FORMWORK REMAINING FORM PREVIOUS CONSTRUCTION AT BACKWALLS AND DIAPHRAGMS SHALL BE REMOVED.
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BAR NUMBER	SPLICE
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20	800
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A) CONCRETE CAST AGAINST EARTH	75mm
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C) DECK – TOP REINFORCING	65mm
D) DECK – BOTTOM REINFORCING	30mm
E) CURB REINFORCING	65mm

No.	By	REVISION DESCRIPTION	Date

FOR USE BY TRAFFIC ENGINEERING ONLY

Designed By	Date	Checked By	Date

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET WHERE TAKEN
C. SHEET WHERE DETAILED

DO NOT SCALE DRAWINGS

Project/Drawing Title

PART "C"
CANNING BRIDGE

EXISITNG AND PROPOSED NORTH ABUTMENT DETAILS

Drawn By: SN
Dwg Chk'd By
Designed By
Des Chk'd By
Approved By
Date: 05-25-10

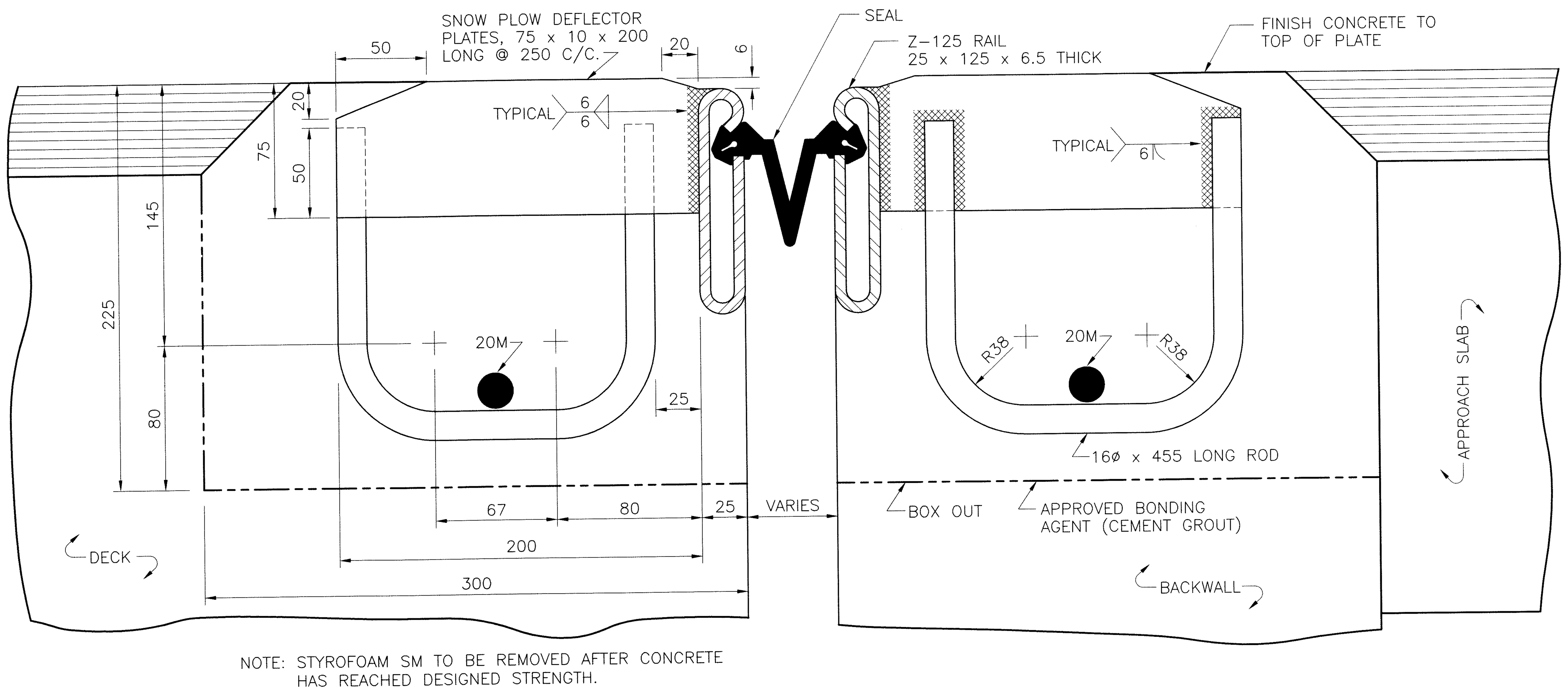
Design Engineer

REGISTERED PROFESSIONAL ENGINEER
R. G. MATTHEWS
07/08/10
PROVINCE OF NEWFOUNDLAND

" FOR CONSTRUCTION "

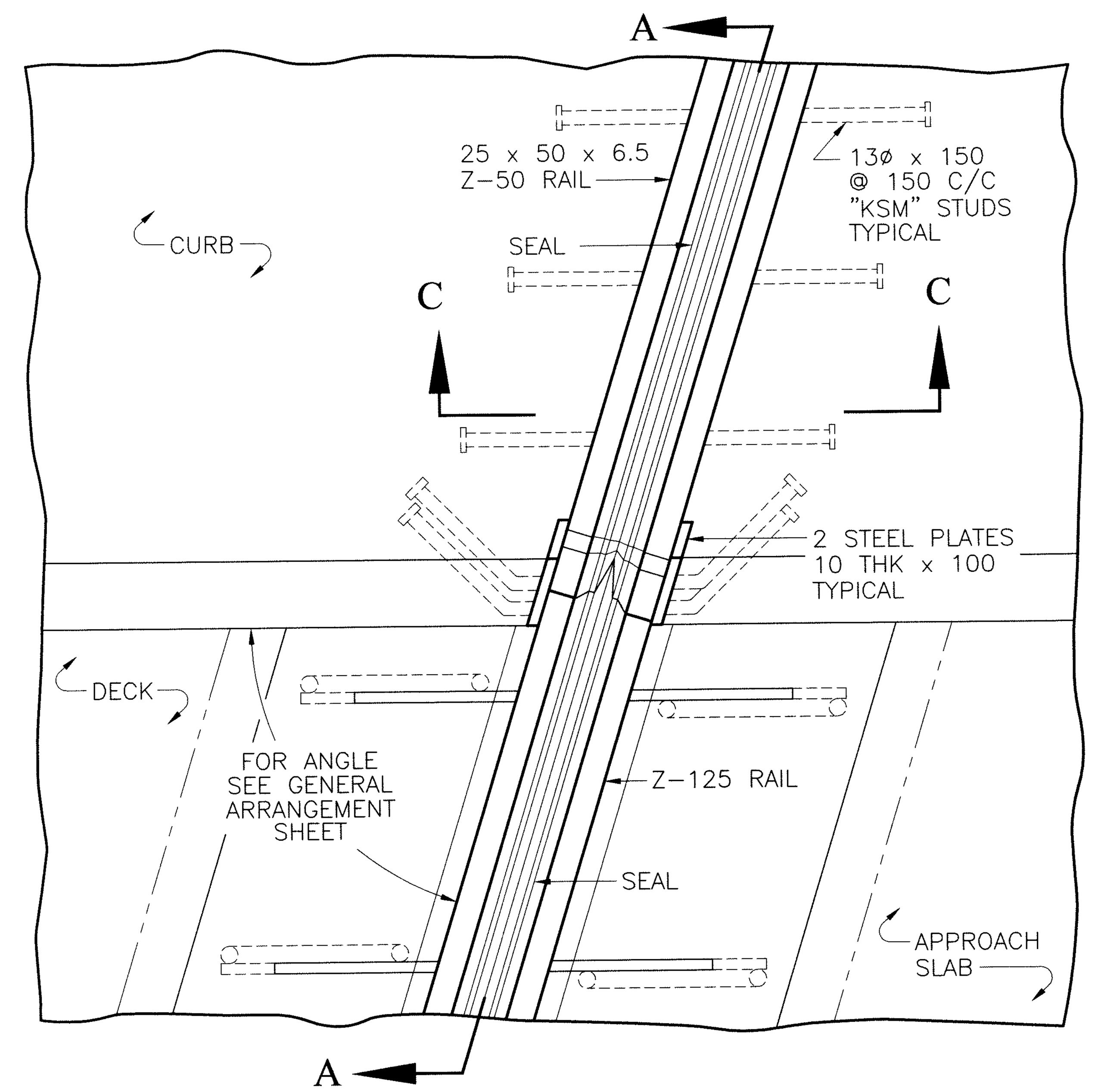
Project No. 35-10 PSR

Sheet No. 6 of 8

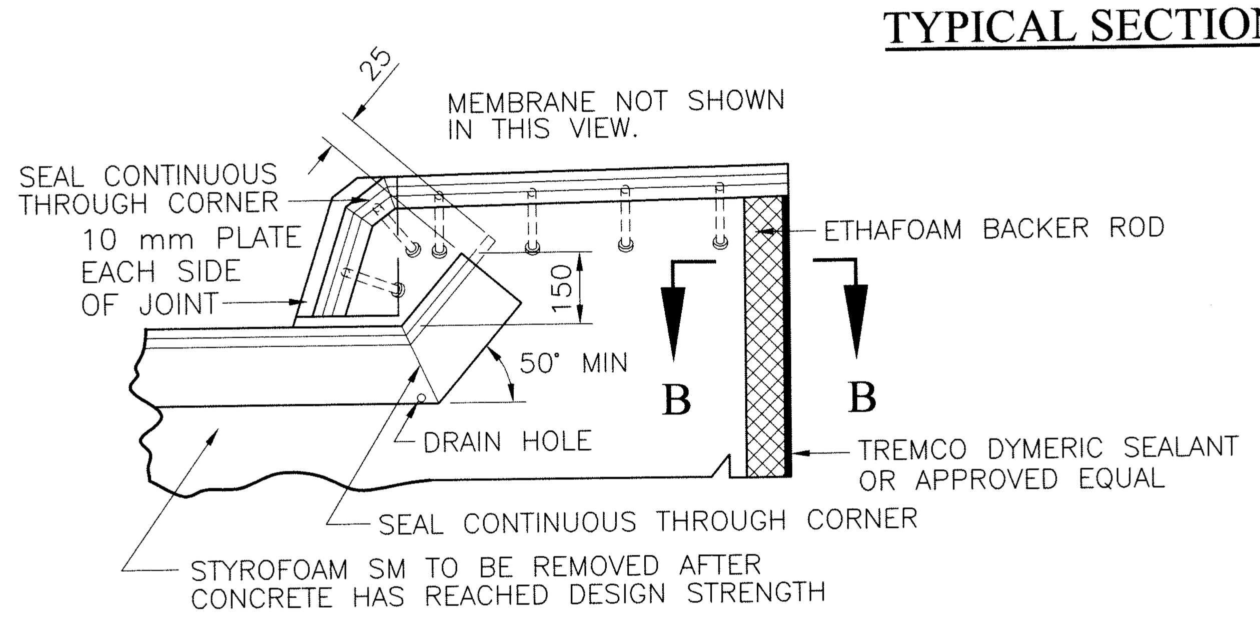


NOTE: STYROFOAM SM TO BE REMOVED AFTER CONCRETE HAS REACHED DESIGNED STRENGTH.

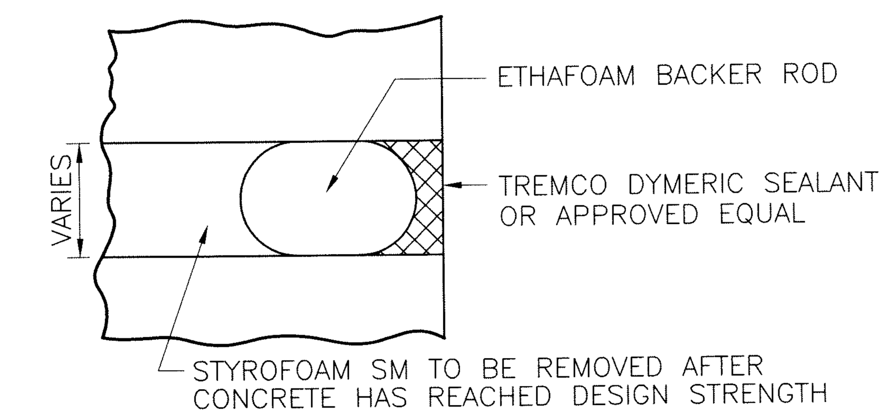
TYPICAL SECTION FOR DECK EXPANSION JOINT
NOT TO SCALE



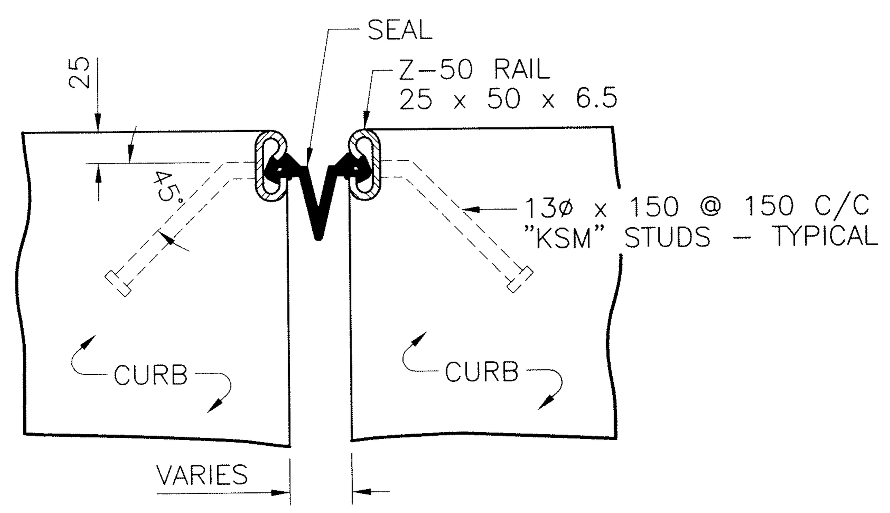
EXPANSION JOINT PLAN
NOT TO SCALE



SECTION A-A
NOT TO SCALE

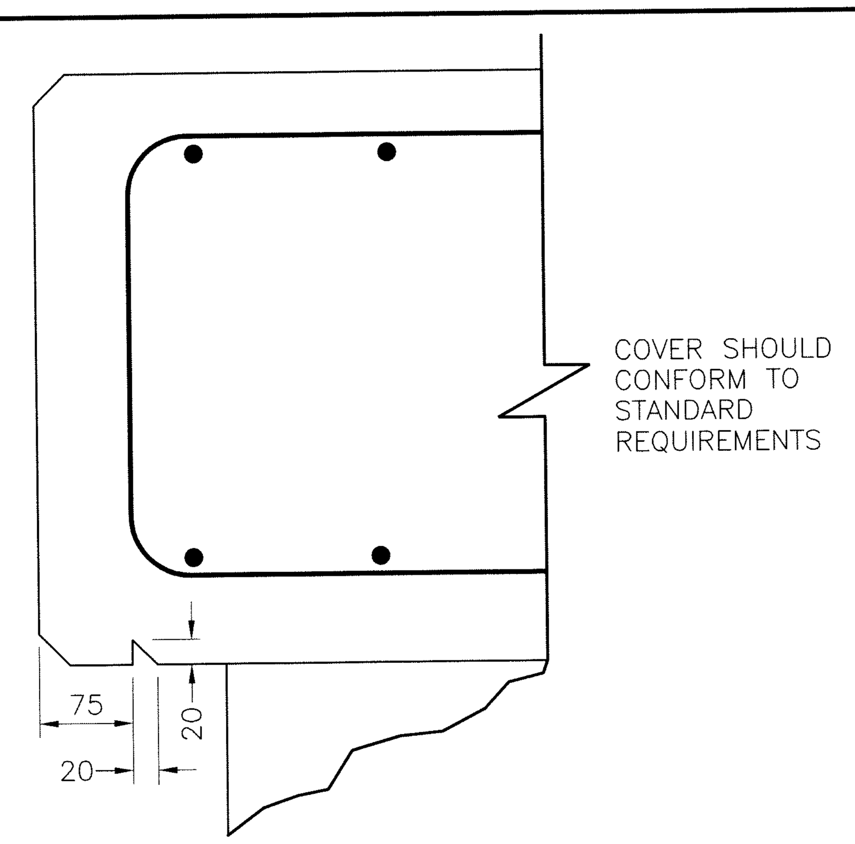


SECTION B-B
NOT TO SCALE



SECTION C-C
NOT TO SCALE

APPROVED LIST
Z-TECH ACME GANAWA WATSON BOWMAN ACME



DRIP BEAD DETAIL (FOR USE ON CANNING BRIDGE)
NOT TO SCALE

Site No. 2-003, 2-044

Newfoundland Labrador
TRANSPORTATION AND WORKS
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

1. MAXIMUM BOXOUT DIMENSIONS 225 mm DEEP x 300 mm WIDE.
2. CONFLICTS WITH PRESTRESS STEEL ANCHORAGES MUST BE REPORTED TO THE ENGINEER.
3. FOR POST-TENSIONED SLAB TYPE STRUCTURES, REFER TO LONGITUDINAL POST-TENSIONING DRAWINGS FOR ANCHOR LOCATION AND BOX-OUT DETAILS.
4. SNOW PLOW PLATES TO BE PLACED PARALLEL TO THE CENTERLINE OF ROADWAY.
5. RATTLE BROOK BRIDGE GAP TO BE 50 mm @ 15°C (DURING INSTALLATION). MOVEMENT = 20 mm.
6. CANNING BRIDGE GAP TO BE 50 mm @ 15°C (DURING INSTALLATION). MOVEMENT = 20 mm.

No.	By	REVISION DESCRIPTION	Date

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET WHERE TAKEN
C. SHEET WHERE DETAILED

DO NOT SCALE DRAWINGS

Project/Drawing Title

PARTS "A" & "C"
RATTLE BROOK AND
CANNING BRIDGE

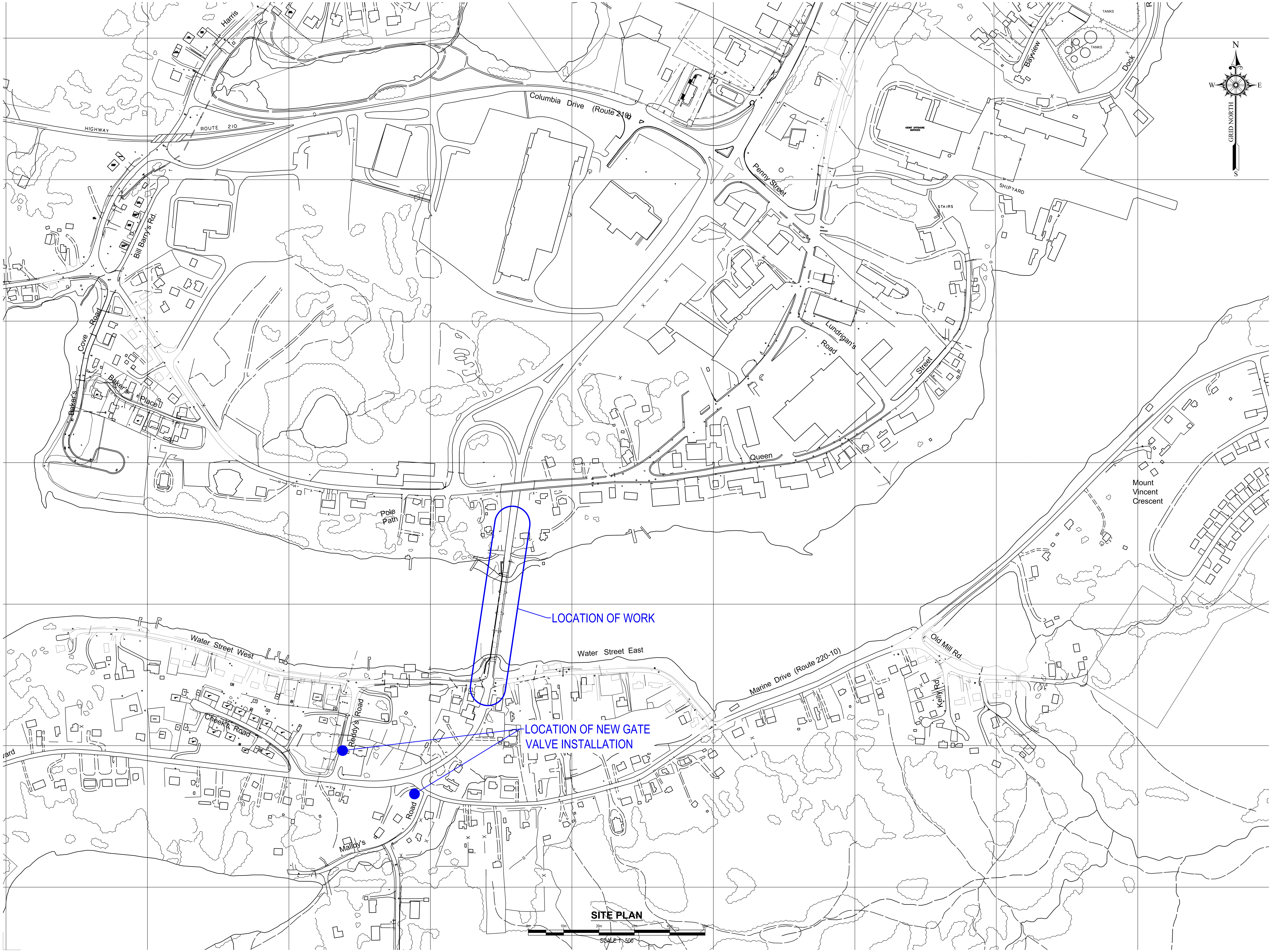
EXPANSION JOINT AND DRIP BEAD DETAIL

Drawn By: J.C. Roberts
Dwg Chk'd By:
Designed By:
Des Chk'd By:
Approved By:
Date: 05-13-10
Design Engineer:

" FOR CONSTRUCTION "

Project No. 35-10 PSR

Sheet No. 7 of 8



CAUTION: DO NOT SCALE DRAWINGS.

No.	Issue	Date
A	ISSUED FOR APPROVAL	21/JULY/2017
T	ISSUED FOR TENDER	28/JULY/2017
C	ISSUED FOR CONSTRUCTION	08/SEPT/2017
AB	AS-BUILT	02/MAY/2018

No.	Revision	Date

CAP
CAP Management Services
120 Conception Bay Highway
Suite 108, Villa Nova Plaza
Conception Bay South, NL | A1W 3A6
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www.capms.ca

AS-BUILT

Professional Seal(s)

REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
PROFESSIONAL ENGINEERS AND GEOSCIENTISTS
STEVEN FRIZZELL
Steven Frizzell
SIGNATURE
Sept. 8/2017
DATE
NEWFOUNDLAND AND LABRADOR

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PROFESSIONAL ENGINEERS AND GEOSCIENTISTS
PERMIT HOLDER
This Permit Allows
CAP MANAGEMENT SERVICES LTD.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL N0818
which is valid for the year 2017
Member in Responsible Charge 08253

Drawn By:	G.J.B.R.
Dwg Standards Ckd By:	C.R.M.
Designed By:	C.R.M.
Design Approved By:	S.F.
Scale:	SCALE AS NOTED

Client Title

TOWN OF MARYSTOWN
NEWFOUNDLAND AND LABRADOR

Project Title

CANNING BRIDGE WATER MAIN REPLACEMENT

Dwg. Title

LOCATION PLAN

Project No.
NL00007

MAE No.
17-MYCW-18-00123

Dwg. No.	Rev. No.
G1	0

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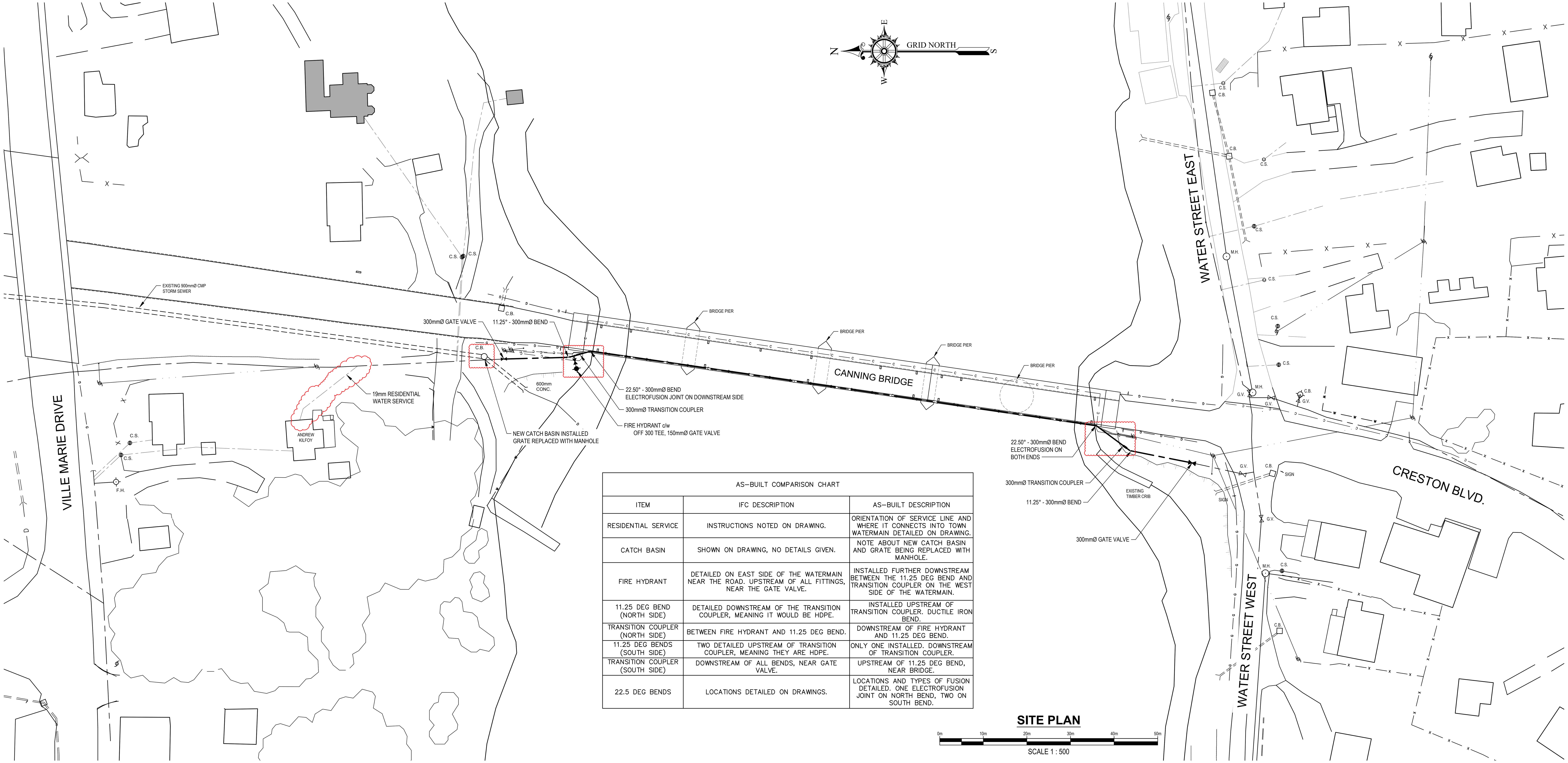
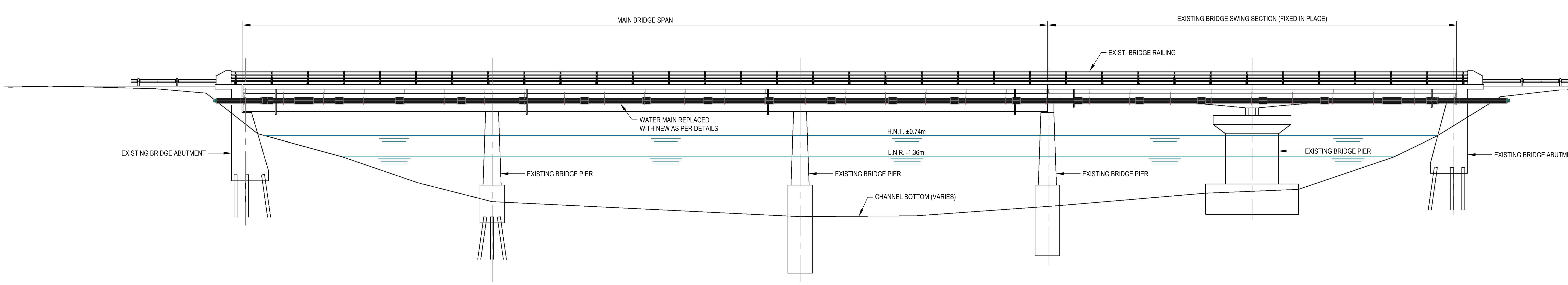
AB	AS-BUILT	02/MAY/2018



AS–BUILT COMPARISON CHART		
ITEM	IFC DESCRIPTION	AS–BUILT DESCRIPTION
RESIDENTIAL SERVICE	INSTRUCTIONS NOTED ON DRAWING.	ORIENTATION OF SERVICE LINE AND WHERE IT CONNECTS INTO TOWN WATERMAIN DETAILED ON DRAWING.
CATCH BASIN	SHOWN ON DRAWING, NO DETAILS GIVEN.	NOTE ABOUT NEW CATCH BASIN AND GRATE BEING REPLACED WITH MANHOLE.
FIRE HYDRANT	DETAILED ON EAST SIDE OF THE WATERMAIN NEAR THE ROAD. UPSTREAM OF ALL FITTINGS, NEAR THE GATE VALVE.	INSTALLED FURTHER DOWNSTREAM BETWEEN THE 11.25 DEG BEND AND TRANSITION COUPLER ON THE WEST SIDE OF THE WATERMAIN.
11.25 DEG BEND (NORTH SIDE)	DETAILED DOWNSTREAM OF THE TRANSITION COUPLER, MEANING IT WOULD BE HDPE.	INSTALLED UPSTREAM OF TRANSITION COUPLER. DUCTILE IRON BEND.
TRANSITION COUPLER (NORTH SIDE)	BETWEEN FIRE HYDRANT AND 11.25 DEG BEND.	DOWNSTREAM OF FIRE HYDRANT AND 11.25 DEG BEND.
11.25 DEG BENDS (SOUTH SIDE)	TWO DETAILED UPSTREAM OF TRANSITION COUPLER, MEANING THEY ARE HDPE.	ONLY ONE INSTALLED. DOWNSTREAM OF TRANSITION COUPLER.
TRANSITION COUPLER (SOUTH SIDE)	DOWNSTREAM OF ALL BENDS, NEAR GATE VALVE.	UPSTREAM OF 11.25 DEG BEND, NEAR BRIDGE.
22.5 DEG BENDS	LOCATIONS DETAILED ON DRAWINGS.	LOCATIONS AND TYPES OF FUSION DETAILED. ONE ELECTROFUSION JOINT ON NORTH BEND, TWO ON SOUTH BEND.

SITE PLAN

SCALE 1 : 500



BRIDGE ELEVATION

SCALE 1 : 250

No.	Revision	Date

CAP

120 Conception Bay Highway
Suite 108, Villa Nova Plaza
Conception Bay South, NL | A1W 3A6
T: 1 877 757 7204 | F: 709 782 8941
www.capms.ca

AS-BUILT

Professional Seal(s)

REGISTERED PROFESSIONAL ENGINEER – PEG – STEVEN FRIZZELL – SIGNATURE – Sept. 8/2017 – DATE – NEWFOUNDLAND AND LABRADOR

PROVINCE OF NEWFOUNDLAND AND LABRADOR – PEG – PERMIT HOLDER – This Permit Allows CAP MANAGEMENT SERVICES LTD. To practice Professional Engineering in Newfoundland and Labrador. Permit No. as issued by PEGNL N0818 which is valid for the year 2017. Member in Responsible Charge 08253

Drawn By:	G.J.B.R.
Dwg Standards Ckd By:	C.R.M.
Designed By:	C.R.M.
Design Approved By:	S.F.
Scale:	SCALE AS NOTED

Client Title

TOWN OF MARYSTOWN
NEWFOUNDLAND AND LABRADOR

Project Title

CANNING BRIDGE WATER MAIN REPLACEMENT

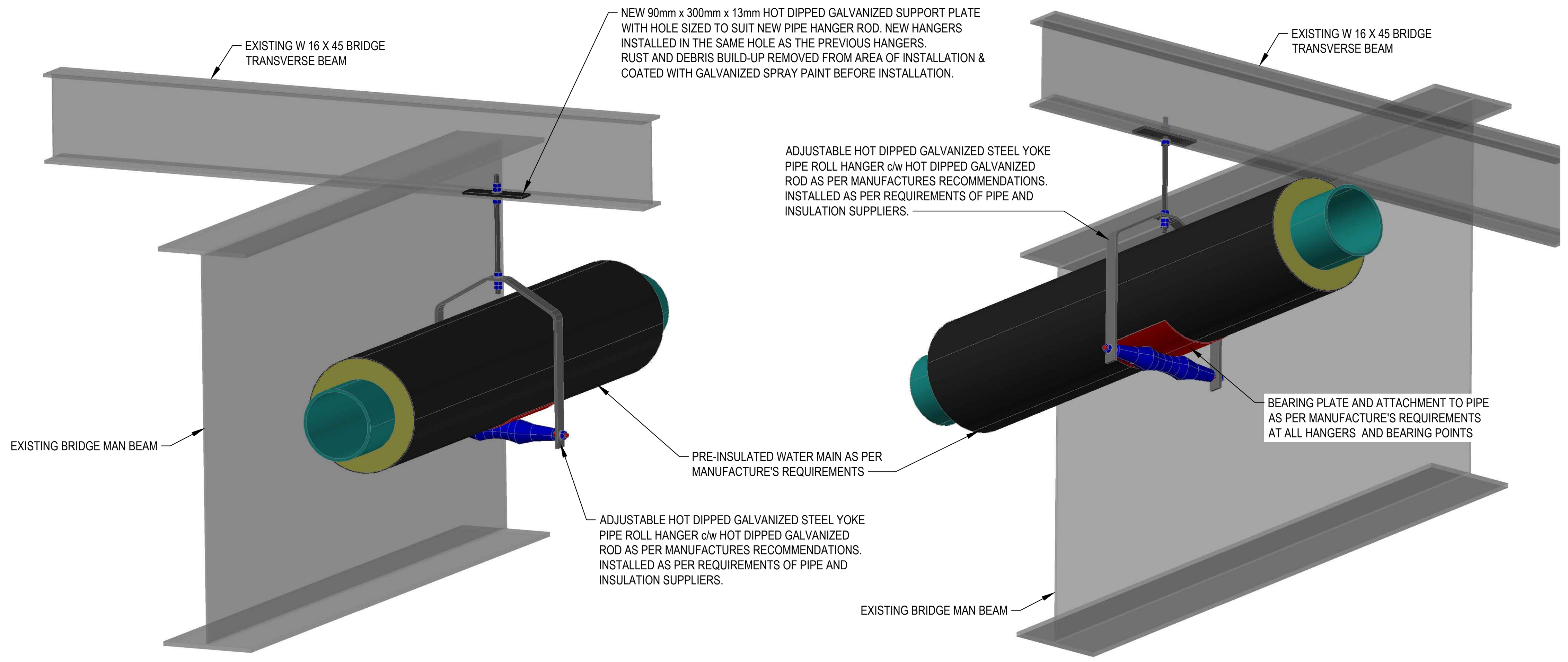
Dwg. Title

SITE PLAN AND ELEVATION

Project No. **NL00007**

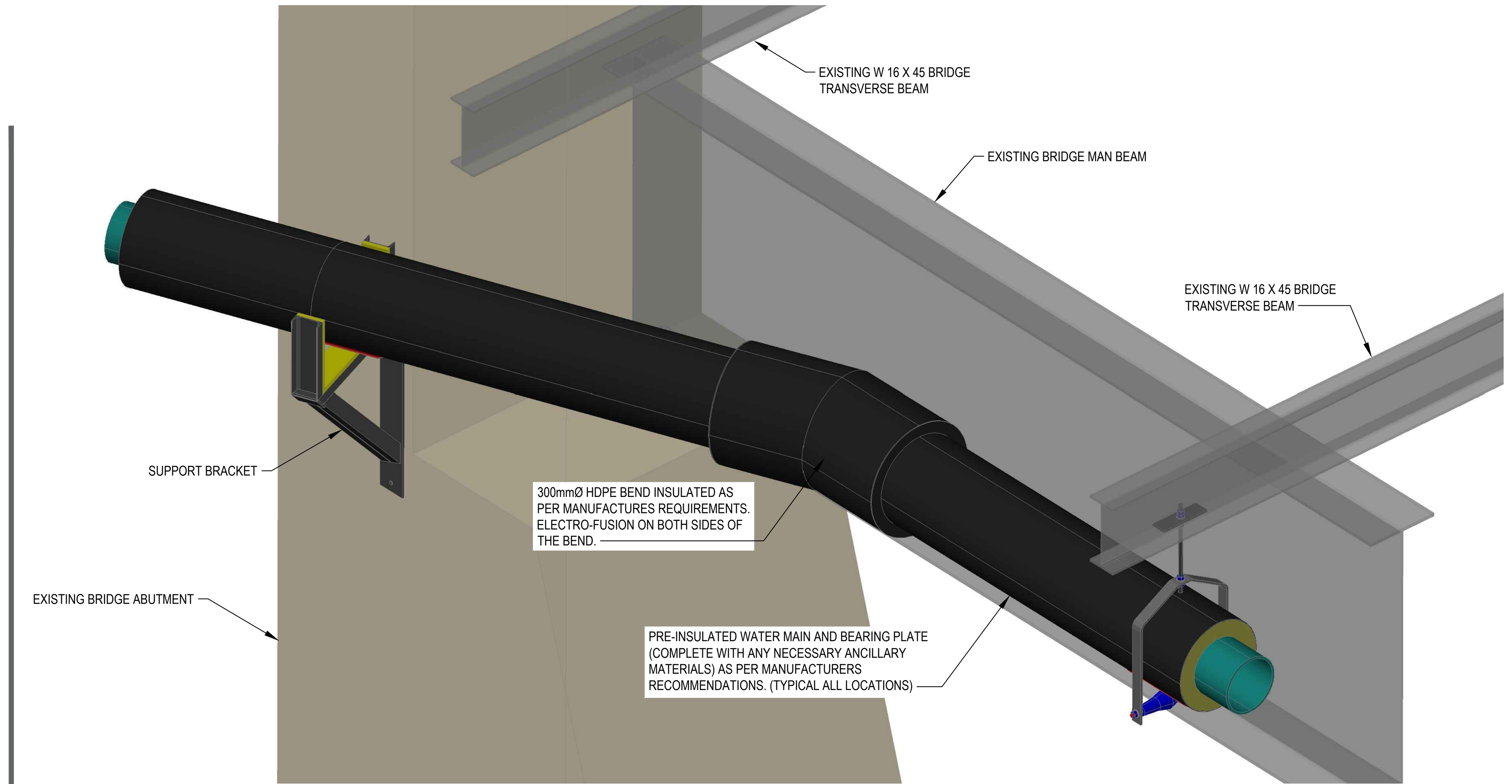
CWWF No. **17-MYCW-18-00123**

Dwg. No. **C1** Rev. No. **0**



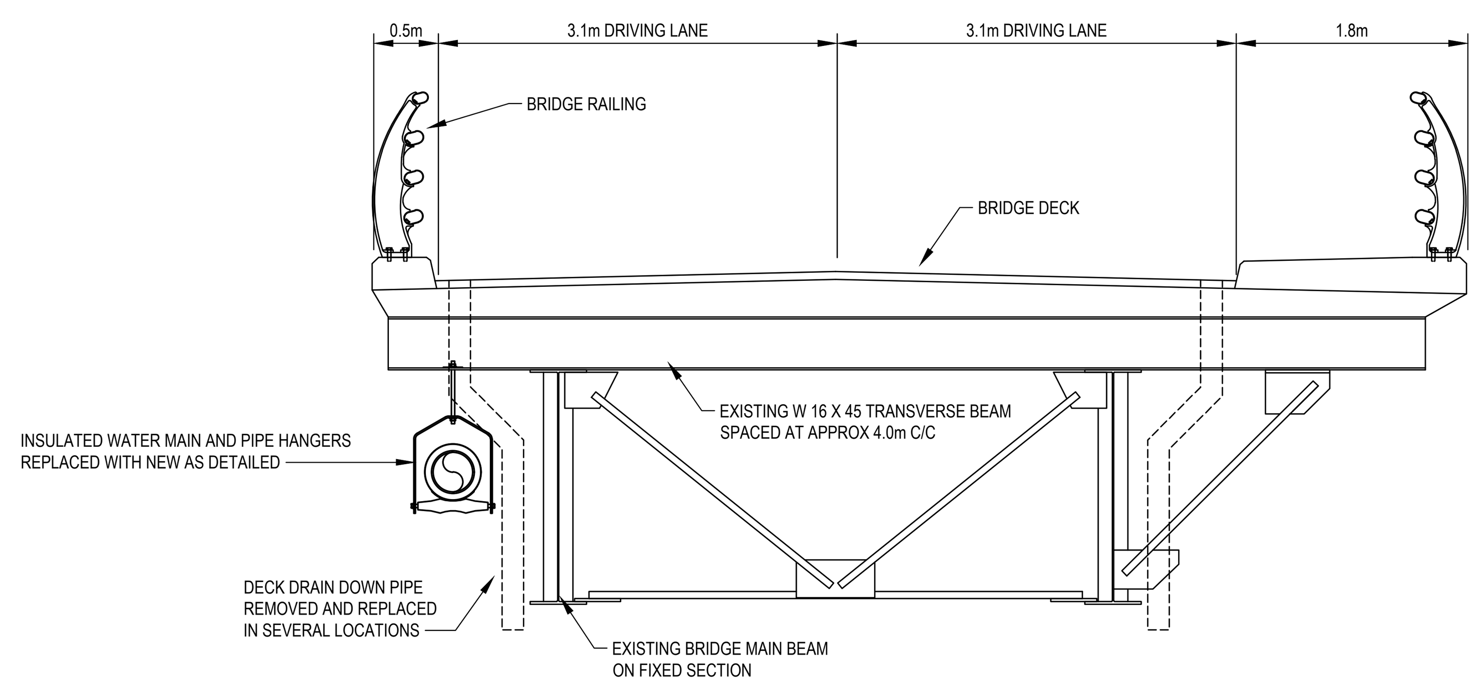
PIPE HANGER DETAIL

SCALE 1 : 20



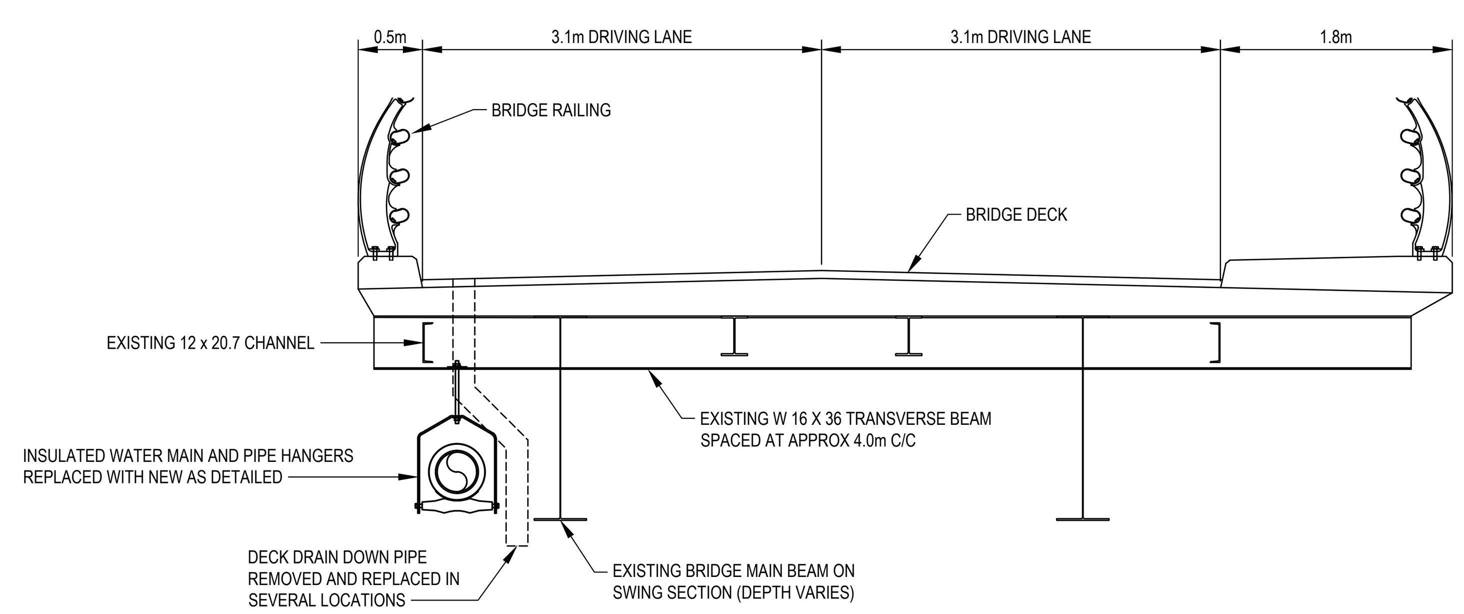
PICTORIAL VIEW

SCALE 1 : 25



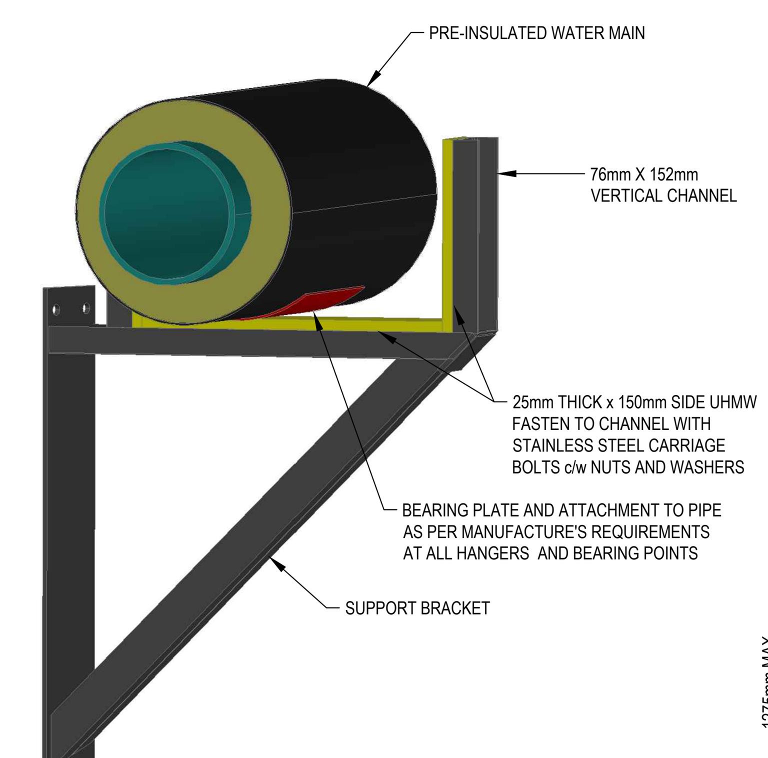
BRIDGE SECTION (FIXED SECTION)

SCALE 1 : 50

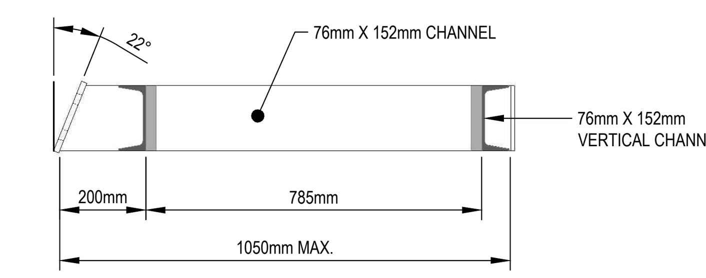


BRIDGE SECTION (SWING SECTION)

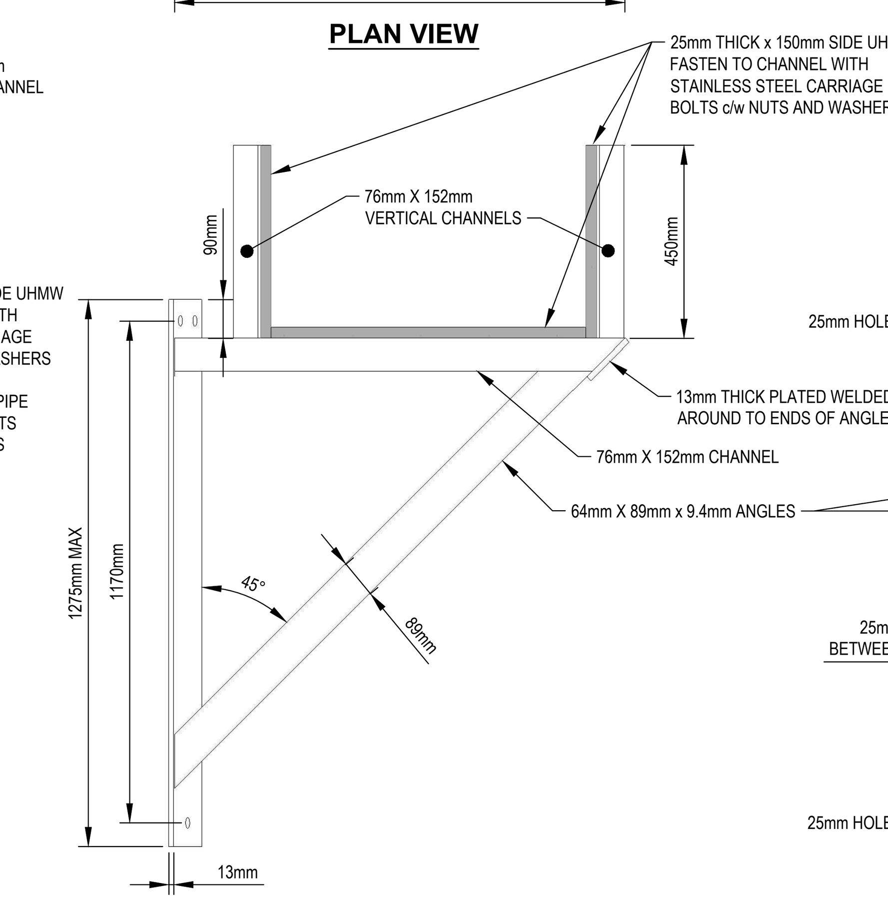
SCALE 1 : 50



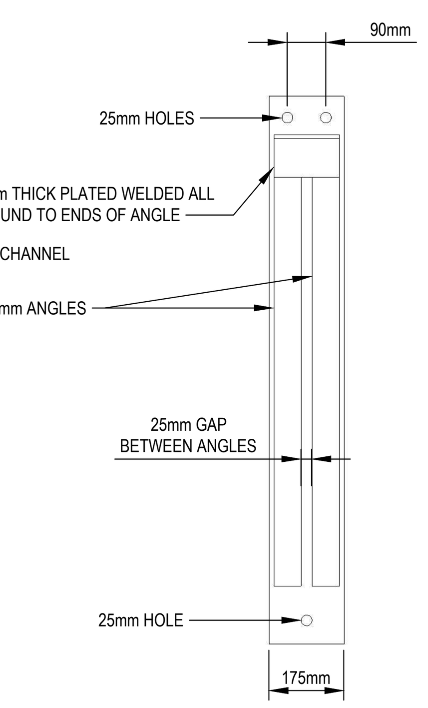
NOTE
BRACKET ATTACHED TO EXISTING CONCRETE ABUTMENTS WITH APPROVED CONCRETE ANCHORS DESIGNED TO SUPPORT THE LOAD OF THE PIPE ASSEMBLY AND WATER.



PLAN VIEW



SIDE VIEW



END VIEW

PIPE HANGING DETAIL AT BRIDGE ABUTMENT

SCALE 1 : 15

CAUTION: DO NOT SCALE DRAWINGS.

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AB	AS-BUILT	02/MAY/2018

No.	Revision	Date

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www.capms.ca

AS-BUILT

Professional Seal(s)

REGISTERED PROFESSIONAL ENGINEER
PEG
STEVEN FRIZZELL
SIGNATURE
Sept. 8/2017
DATE
NEWFOUNDLAND AND LABRADOR

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PEG PERMIT HOLDER
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Member in Responsible Charge 08253

Drawn By:	G.J.B.R.
Dwg Standards Ckd By:	C.R.M.
Designed By:	C.R.M.
Design Approved By:	S.F.
Scale:	SCALE AS NOTED

Client Title

TOWN OF MARYSTOWN
NEWFOUNDLAND AND LABRADOR

Project. Title

CANNING BRIDGE WATER MAIN REPLACEMENT

Dwg. Title

DETAILS

Project No.
NL00007

MAE No.
17-MYCW-18-00123

Dwg. No. **C2**
Rev. No. **0**

CONSTRUCTION NOTES

GENERAL

- THESE PROJECT DOCUMENTS HAVE BEEN PREPARED IN CONJUNCTION WITH THE STANDARD "MUNICIPAL WATER, SEWER, AND ROADS MASTER CONSTRUCTION SPECIFICATIONS." AS PUBLISHED BY THE PROVINCE OF NEWFOUNDLAND AND LABRADOR, DEPARTMENT OF MUNICIPAL AFFAIRS. AND ALSO WITH THE TRANSPORTATION AND WORKS (TW) SPECIFICATIONS BOOK AS PUBLISHED BY THE DEPARTMENT OF TRANSPORTATION AND WORKS HIGHWAY DESIGN DIVISION 3.
- CONTRACTOR TO ENSURE THAT ALL WORK IS CARRIED OUT AND COMPLY WITH THE OCCUPATIONAL HEALTH AND SAFETY ACT.
- THE CONTRACTOR IS RESPONSIBLE FOR NOTIFYING THE ENGINEER IF WORK IS SUSPENDED FOR ANY PERIOD OF TIME AFTER INITIAL START-UP. THE CONTRACTOR SHALL NOTIFY THE ENGINEER FORTY-EIGHT (48) HOURS PRIOR TO RESTART.
- DURING THE COURSE OF CONSTRUCTION OF THE PROJECT THE CONTRACTOR SHALL BE SOLELY AND COMPLETELY RESPONSIBLE FOR CONDITIONS AT AND ADJACENT TO THE WORK SITE INCLUDING SAFETY OF ALL PERSONS SAND PROPERTY DURING PERFORMANCE OF THE WORK. THE CONTRACTOR SHALL PROVIDE ALL LIGHTS, SIGNS, BARRICADES, FLAG PERSONS, AND / OR OTHER DEVICES NECESSARY TO PROVIDE FOR PUBIC SAFETY.THIS REQUIREMENT SHALL APPLY CONTINUOUSLY AND IS NOT LIMITED TO NORMAL WORKING HOURS.
- THE CONTRACTOR SHALL PERFORM ALL WORK SHOWN ON THE DRAWINGS AND ALL INCIDENTAL WORK CONSIDERED NECESSARY TO COMPLETE THR PROJECT IN AN ACCEPTABLE MANNER.
- THE CONTRACTOR SHALL IMMEDIATELY REMOVE ANY CONSTRUCTION DEBRIS OR MUD TRACKED ONTO ROADWAYS.
- THE CONTRACTOR SHALL REPAIR ANY EXCAVATION OR PAVEMENT FAILURES CAUSED BY HIS CONSTRUCTION.
- THE CONTRACTOR SHALL BE RESPONSIBLE FOR REMOVING ANY GROUNDWATER ENCOUNTERED DURING CONSTRUCTION OF ANY PORTION OF THE PROJECT. GROUNDWATER SHALL BE PUMPED, PIPED REMOVED AND DISPOSED OF IN A MANNER WHICH DOES NOT CAUSE FLOODING OF NEITHER EXISTING STREET NOR EROSION OF ABUTTING PROPERTIES. THE USE OF ANY SANITARY SEWER TO DISPOSE OF TRENCH WATER WILL NOT BE PERMITTED.
- ALL SITE WORK SHALL BE CONSTRUCTED IN ACCORDANCE WITH THE APPROVED PLANS. ANY DEVIATION FROM THE APPROVED PLANS WILL REQUIRE PRIOR APPROVAL FROM THE OWNER, THE ENGINEER, AND OTHER APPROPRIATE PUBLIC AGENCIES.
- THE LOCATION OF ALL EXISTING INFRASTRUCTURE SUCH AS WATER, SANITARY SEWER AND STORM SEWER MAINS AND ALL OTHER INFRASTRUCTURE IS SHOWN IN APPROXIMATE LOCATION AND MAY NOT BE NECESSARILY BE COMPLETE. IT IS THE SOLE RESPONSIBILITY OF THE CONTRACTOR TO INDEPENDENTLY VERIFY EXACT LOCATION OF ALL EXISTING INFRASTRUCTURE AND ADVICE THE ENGINEER OF ANY DISCREPANCIES PRIOR TO START OF CONSTRUCTION.
- THE CONTRACTOR SHALL LOCATE AND PROTECT ALL UTILITIES DURING CONSTRUCTION AND SHALL CONTACT THE UNDERGROUND UTILITIES COMPANIES TO DETERMINE THE LOCATE OF ALL UNDERGROUND UTILITIES AT LEAST FORTY-EIGHT (48) HOURS PRIOR TO START OF CONSTRUCTION.
- BENCH MARK IS BASED ON GEODETIC DATUM.
- ALL SURVEY CO-ORDINATES AND ELEVATIONS ARE BASED ON THE MODIFIED THREE DEGREE TRANSVERSE MERCATOR PROJECTION, ZONE 2, NAD 83, FOR THE PROVINCE OF NEWFOUNDLAND AND LABRADOR UNLESS OTHERWISE NOTED.
- ALL ELEVATIONS AND CHAINAGES IN METRES UNLESS OTHERWISE NOTED. ALL DIMENSIONS IN MILLIMETRES.
- A COPY OF THE APPROVED CONSTRUCTION PLANS MUST BE ON THE JOB SITE WHEN CONSTRUCTION IS IN PROGRESS.
- THE CONTRACTOR SHALL EXCAVATE TEST HOLES PRIOR TO PIPE INSTALLATION, AND CONFIRM WITH THE ENGINEER, PIPE DEPTH AND LOCATION OF ALL UNDERGROUND PIPES IN AREAS WHERE NEW PIPE MUST CROSS THESE UTILITIES.
- PRIOR TO COMMENCING ANY CONSTRUCTION, THE CONTRACTOR SHALL CARRY OUT DETAILED SURVEYS, INCLUDING CROSS-SECTIONS. ALL SURVEYS TO BE CONFIRMED WITH THE ENGINEER.
- THE CONTRACTOR SHALL RESTORE ALL DISTURBED AREAS TO EXISTING CONDITIONS OR BETTER TO THE SATISFACTION OF THE ENGINEER.

WATER AND SEWER

- WHERE TEMPORARY WATER SERVICING IS REQUIRED UNDER THE CONTRACT THE CONTRACTOR SHALL SUPPLY, INSTALL AND MAINTAIN TEMPORARY WATER SERVICES WHERE REQUIRED TO PROPERTIES. ALL TEMPORARY SERVICE CONNECTION MATERIALS SHOULD CONFORM TO THE NSF/ANSA STANDARD 61. ALL HOSE USED FOR INDIVIDUAL PROPERTY CONNECTIONS SHALL BE A MINIMUM 19mm INTERNAL DIAMETER, DESIGNED FOR A WORKING PRESSURE OF 860 kPa AND BE FREE FREE FROM DEFECTS IN MATERIAL AND WORKMANSHIP. THE PIPE, HOSE AND ALL OTHER MATERIALS WHICH ARE TO BE FURNISHED BY THE CONTRACTOR FOR USE IN CONJUNCTION WITH THE TEMPORARY SERVICE PIPE AND TEMPORARY CONNECTIONS TO PROPERTY SERVICES AND BRANCHES SHALL BE APPROVED BY THE ENGINEER, AND SHALL BE OF MATERIAL WHICH DOES NOT IMPART AND TASTE OR ODOUR TO THE WATER IN ACCORDANCE WITH NSF/ANSI STANDARD 61.
- TEMPORARY WATER SERVICING PIPE AND FITTINGS SHALL PROVIDE ADEQUATE WATER TIGHTNESS AND CARE SHALL BE EXERCISED THROUGHOUT THE INSTALLATION OF TEMPORARY PIPE AND SERVICE FITTINGS TO AVOID THE POSSIBLE OF ANY TOWN MAIN OR PROPERTY SERVICES OR CONTAMINATION OF THE TEMPORARY SERVICE PIPE PROPER. FLUSHING OF THE PRIVATE SERVICE CONNECTIONS AND CHLORINATION OF THE BYPASS LINE PRIOR TO THEIR USE WILL BE REQUIRED.
- HOUSE SERVICES SHALL EXTEND TO 300mm FROM PROPERTY BOUNDARY OR ROAD RIGHT-OF -WAY UNLESS OTHER WISE DIRECTED BY THE ENGINEER..
- CONTRACTOR SHALL CONFIRM INVERTS AND LOCATION OF ALL WATER AND SEWER SERVICE LATERALS WITH THE ENGINEER PRIOR TO INSTALLATION IN FIELD.
- ALL WATER MAIN STUBS TO BE 5.5m MIN. BEYOND VALVE. LOCATION SHOWN FOR VALVES, HYDRANTS, TEES AND CAPS ON DRAWING ARE APPROXIMATE. EXACT LOCATIONS WILL BE FIELD DETERMINED WITH THE ENGINEER.
- EACH WATER MAIN END CAP TO HAVE CONCRETE THRUST BLOCK.
- MINIMUM LENGTH OF ALL WATER MAIN SHORT PIECES AT FITTINGS TO BE 1.0m ON LESS OTHER WISE NOTED OR DIRECTED BY THE ENGINEER.
- WATER MAINS TO HAVE MECHANICAL JOINT RESTRAINTS ON STUBS AND LAST 18.0m OF DEAD ENDS IN CONJUNCTION WITH THRUST BLOCKS
- EACH VALVED SECTION OF WATER MAIN TO BE TESTED AND MADE READY FOR OPERATION IMMEDIATELY FOLLOWING INSTALLATION OF THAT SECTION.
- ALL WATER MAIN PIPES TO BE PRESSURE CLASSED AS SPECIFIED IN THE SCHEDULE OF QUANTITIES AND PRICES.
- GRADES AND INVERTS OF ALL SEWER STUBS SHALL BE CONFIRMED IN FIELD WITH THE ENGINEER.
- ALIGNMENT FOR ALL WATER AND SEWER STUBS SHALL BE CONFIRMED WITH THE ENGINEER IN FIELD PRIOR TO INSTALLATION.
- THE BONNETS (TOPS) AND NOZZLE CAPS OF HYDRANTS TO BE PAINTED WITH THE FOLLOWING IN COLOUR SCHEME INDICATING CAPACITY.

COLOUR FLOW CAPACITY (LPS)	COLOUR	
114 OR MORE	LIGHT BLUE	
75 TO 114	GREEN	(CONFIRM COLOUR
38 TO 75	ORANGE	WITH ENGINEER FOR
LESS THAN 38	RED	ALL FIRE HYDRANTS)

PAINT SPECIFICATION; ONE COAT MATCHLESS STRUCTURAL STEEL PRIMER (13-110) AND TWO COATS MATCHLESS SUPERMARINE ENAMEL 700 SERIES OR APPROVED EQUAL

- FIRE HYDRANT INSTALLATION, LOCATION, SPACING AND COLOUR TO CONFORM TO FIRE COMMISSIONER'S BULLETIN #11 DATED 2000-02-16 OR NEWER ENTITLED "FIRE HYDRANTS, FIRE HOSE THREADS AND WATER SUPPLIES."
- ALL MANHOLE FRAMES, COVERS AND VALVE BOX COVERS ON STREETS TO BE PAVED OR RESURFACED SHALL BE ADJUSTED TO FINISHED ROAD GRADE.
- WATER SERVICE LATERALS FROM THE NEW WATER MAIN TO THE CURB STOP ARE TO BE CONTINUOUS WITHOUT JOINTS (JOINERS ARE NOT TO BE USED).
- ALL MANHOLES LOCATED OUTSIDE OF THE STREET R.O.W. ARE TO HAVE BOLT DOWN TYPE FRAME AND COVERS.
- ALL WATER AND SEWER MAINS THAT HAVE BEEN INSTALLED ARE TO BE TESTED, APPROVED AND MADE READY FOR USE PRIOR TO THE CONTRACTORS SEASONAL SHUT DOWN AT YEARS END.
- WHERE FIRE HYDRANT LEADS CROSS ROAD DITCHES, 3.0m OF CULVERT IS TO BE INSTALLED OVER THE HYDRANT LEAD TO PERMIT IN FILLING OF THE DITCH. THE HYDRANT LEAD IS TO BE INSULATED 1.2m EACH SIDE OF THE DITCH CENTER LINE BY PLACING 100mm OF RIGID STYROFOAM 'SM' INSULATION, 300mm ABOVE LEAD. THE PLACING OF THE INSULATION IS TO BE INCLUDED IN THE PRICE FOR 150mm WATER MAIN. (SEE DETAIL DWG. No. ???).
- ALL SEWER OUTFALLS AND WATER SUPPLY INTAKES TO BE T.V. INSPECTED AND COMPLETE VIDEO TAPE TO BE SUBMITTED TO THE ENGINEER. PAYMENT TO BE AS PER ITEM IN SCHEDULE OF QUANTITIES AND PRICES.
- ON ALL STREETS, THE CONTRACTOR SHALL VERIFY WITH THE TOWN AND THE ENGINEER, LOCATIONS OF HOUSE SERVICE LATERALS PRIOR TO EXCAVATION.
- THE CONTRACTOR IS TO ADVISE ALL HOMES CURRENTLY SERVICED WITH WATER AND SEWER TWENTY-FOUR HOUR (24) IN ADVANCE WHEN THEIR SERVICES ARE TO BE DISRUPTED. THE WORK IS TO BE SCHEDULED, COORDINATED AND METHODS USED TO MINIMIZE DISRUPTION TO THESE SERVICES. IF DISRUPTION IS UNAVOIDABLE AND SERVICES MUST BE MAINTAINED THEN THE WORK AND / OR MATERIALS REQUIRED TO MAINTAIN THESE SERVICES WILL BE PAID FOR UNDER APPROPRIATE CASH ALLOWANCE.
- NEW WATER SYSTEM, INCLUDING HOUSE SERVICES, SHALL BE INSTALLED AND TESTED PRIOR TO CONNECTION TO SUPPLY MAINS. EXISTING HOUSE SERVICES SHALL BE LOCATED AS WORK PROGRESSES AND NEW WATER SERVICE LINES SHALL BE INSTALLED TO NEW CURB STOPS AT THE PROPERTY LINES. FOLLOWING ACCEPTANCE, BY THE ENGINEER, OF NEW SYSTEM, THIS SYSTEM TO BE MADE OPERATIONAL BY COMPLETING ABOVE CONNECTIONS TO SUPPLY MAINS. INDIVIDUAL HOUSE SERVICES SHALL THEN BE CONNECTED AT PROPERTY LINES TO THE NEW SERVICES. ALL PRESSURIZED CONNECTIONS SHALL BE INSPECTED BY THE ENGINEER PRIOR TO BACKFILLING. DISRUPTIONS IN WATER SUPPLY BE LIMITED TO ONLY THAT PERIOD REQUIRED TO INDIVIDUALLY CONNECT EXISTING SERVICE LINES AT PROPERTY LINES TO NEW SERVICES. THE CONTRACTOR SHALL BE RESPONSIBLE FOR MAINTAINING EXISTING WATER SYSTEM IN OPERATION DURING THE CONSTRUCTION OF THE WORK. CONTRACTOR SHALL INSTALL TEMPORARY PLUGS AND THRUST BLOCKS AS REQUIRED FOLLOWING INSTALLATION OF WATER MAIN AND FITTINGS AND PRIOR TO CONNECTION TO EXISTING WATER MAIN.
- WHEN NEW SANITARY SEWER SYSTEM REQUIRES THE CONNECTION OF EXISTING SEWER SERVICES TO THE NEW SYSTEM DURING CONSTRUCTION HYDROSTATIC WILL NOT REQUIRED. DEFLECTION TESTING WILL BE REQUIRED. INSTALLATION IS TO START FROM THE NEW DOWN STREAM MANHOLE AND THE NEW SEWER SYSTEM IS TO BE KEPT FUNCTIONAL AT ALL TIMES. NEW 100mmØ SEWER SERVICES ARE TO BE INSTALLED IN COMMON TRENCH WITH NEW WATER SERVICES. CONTRACTOR SHALL MAINTAIN THE EXISTING SEWER SYSTEM UNTIL NEW SEWER MAIN INSTALLATION HAS BEEN COMPLETED.
- ALL EXISTING FIRE HYDRANTS THAT ARE REMOVED AND TO BE TRANSPORTED TO TOWN / CITY DEPOT.
- EXISTING CURB BOXES AND STOPS THAT ARE REMOVED ARE TO BE DELIVERED TO TOWN / CITY DEPOT WHEN DIRECTED BY THE ENGINEER.
- ON ALL STREETS WHERE STORM SEWER MAINS CROSS NEW OR EXISTING HOUSE SERVICE LATERALS, CLASS 1 CRUSHED STONE BACKFILL SHALL BE USED TO 150mm OVER LATERALS. CRUSHED STONE SHALL BE CONSOLIDATED BY TAMPING. WHERE STORM SEWER MAINS CROSS WITHIN 600mm OF INVERT OF LATERALS, CONCRETE CRADLES SHALL BE USED.
- ALL CONCRETE TO HAVE COMPRESSIVE STRENGTH OF 30 Mpa AT 28 DAYS.

ROAD WORK

- THE LATEST EDITION OF 'THE DEPARTMENT OF TRANSPORTATION AND WORKS SPECIFICATIONS BOOK' DIVISION 3 - 'SPECIFICATIONS FOR PAVEMENT, SELECTED GRANULAR BASE AND RELATED ITEMS' AS PUBLISHED BY THE DEPARTMENT OF DEPARTMENT OF TRANSPORTATION AND WORKS HIGHWAY DESIGN WILL APPLY TO ALL ASPHALT, GRANULAR AND ASSOCIATED WORKS PLACED ON MUNICIPAL PROJECTS.
- PAVING OF STREETS WILL NOT BE ALLOWED UNTIL AFTER COMPLETION OF ALL REQUIRED TESTING AND INSPECTION OF NEW WATER, SEWER AND STORM SEWER SYSTEMS WITHIN THE ROAD WAY LIMITS.
- UNLESS OTHERWISE NOTED ON THE DRAWINGS, NO CUT OR FILL SLOPES SHALL BE CONSTRUCTED STEEPER THAN 2H : 1V.
- THE CONTRACTOR SHALL BE RESPONSIBLE FOR REMOVING AND REPLACING ANY EXISTING SIGNS, STRUCTURES, FENCES, ETC. ENCOUNTERED DURING CONSTRUCTION AND RESTORING THEM TO THEIR ORIGINAL CONDITION.
- ALL LOOSE, ORGANIC, OTHERWISE DELETERIOUS MATERIALS OR SOFT SPOT(S) ARE TO BE EXCAVATED AND REMOVED FROM THE ROADWAY AND UTILITY TRENCHES IN THE ROADWAY AND REPLACED WITH APPROVED COMPACTED FILL.
- STREET PAVING SHALL NOT BEGIN UNTIL SUB-GRADE COMPACTION TESTS ARE TAKEN AND THE ENGINEER APPROVES THE RESULTS.
- AS SOON AS PRACTICAL AFTER COMPLETION OF ALL PAVING AND GRAVEL SHOULDERS RESURFACING, THE CONTRACTOR SHALL REMOVE ALL DIRT, MUD, ROCK GRAVEL AND OTHER FOREIGN MATERIALS FROM THE PAVED SURFACE AND STORM DRAINAGE SYSTEM
- ALL MANHOLE FRAMES, COVERS AND VALVE BOX COVERS ON STREETS TO BE PAVED OR RESURFACED SHALL BE ADJUSTED TO FINISHED ROAD GRADE.
- WHERE APPROVED BY THE ENGINEER REUSE EXISTING CATCH BASIN FRAME AND COVERS.
- CATCH BASINS LEADS SHALL BE 200mmØ HDPE DUAL WALL WITH A 1.0% (MIN.) SLOPE UNLESS OTHERWISE NOTED. DOUBLE CATCH BASIN STRUCTURES TO HAVE A 300mmØ LEAD UNLESS OTHER WISE NOTED.
- THE CONTRACTOR IS TO ENSURE THAT CATCH BASINS ARE INSTALLED AT LOW POINT OF SAG CURB WORK.

AS BUILTS

- THE CONTRACTOR SHALL MAINTAIN TWO (2) SETS OF “AS- BUILT” PLANS SHOWING ALL FIELD CHANGES AND MODIFICATIONS. IMMEDIATELY AFTER CONSTRUCTION COMPLETION, THE CONTRACTOR SHALL DELIVER BOTH COPIES OF RED-LINED PLANS TO THE ENGINEER.
- THE CONTRACTOR SHALL PROVIDE AS-BUILT SURVEY DATA BASED ON THE MODIFIED THREE DEGREE TRANSVERSE MERCATOR PROJECTION, ZONE 2, NAD 83, FOR THE PROVINCE OF NEWFOUNDLAND AND LABRADOR UNLESS OTHERWISE NOTED THE DATA SHALL CONTAIN NORTHING, EASTING AND ELEVATIONS FOR ALL NEWLY INSTALLED INFRASTRUCTURE UNDER THE CONTRACT. WATER SYSTEMS SHALL INCLUDE SURVEY DATA ON ALL INTAKE STRUCTURES, BENDS, VALVES, TEES, REDUCERS, FIRE HYDRANTS, END CAPS AND CURB STOPS. SANITARY SEWER SYSTEMS SHALL INCLUDE SURVEY DATA ON ALL SANITARY MANHOLES, SEWAGE PUMPING STATION, TREATMENT UNITS, OUTFALLS, CLEANOUTS AND SEWER MAIN AND SERVICE END CAPS. STORM SEWER SYSTEM SHALL INCLUDE SURVEY DATA ON ALL STORM MANHOLES, CATCH BASINS END CAPS, CULVERTS, HEADWALLS, AND STORM SEWER SERVICE DROPS.
- THE CONTRACTOR AND THE SITE ENGINEER SHALL PROVIDE A MINIMUM OF TWO PHYSICAL TIES TO ALL MANHOLES, CLEAN-OUTS, CURB STOPS, VALVES, CHAMBERS AND END CAPS.
- ALL NEW CURB STOPS AND GATE VALVES TO HAVE LOCATION NOTED WITH A MINIMUM OF TWO (2) TIES AND NORTH, EAST AND ELEVATION CO-ORDINATES.

LEGEND

	EXISTING WATER MAIN
	EXISTING WATER LINE SERVICE
	PROPOSED WATER MAIN
	EXISTING SANITARY SEWER
	EXISTING SEWER LINE SERVICE
	PROPOSED SANITARY SEWER
	EXISTING SEWAGE FORCE MAIN
	PROPOSED SEWAGE FORCE MAIN
	EXISTING STORM SEWER
	PROPOSED STORM SEWER
X	EXISTING FENCE
G	EXISTING GUIDE RAIL
G	PROPOSED GUIDE RAIL
D	EXISTING DRAINAGE DITCH
	PROPOSED DRAINAGE DITCH
W	EXISTING RETAINING WALL
	PROPOSED GABION WALL
C	EXISTING BURIED CABLE
	OVERHEAD WIRED
	EXISTING MANHOLES
	PROPOSED MANHOLES
	EXISTING SEWER CLEAN OUT
	PROPOSED SEWER CLEAN-OUT
	EXISTING VALVES
	PROPOSED VALVES
	EXISTING BEND
	NEW BEND
	EXISTING REDUCER
	PROPOSED REDUCER
	EXISTING TEE
	PROPOSED TEE
	EXISTING COUPLER
	NEW COUPLER
	EXISTING END CAP
	PROPOSED END CAP
	EXISTING CURB STOP
	PROPOSED CURB STOP
	EXISTING CULVERT
	PROPOSED CULVERT
	EXISTING CATCH BASINS
	PROPOSED CATCH BASINS
	EXISTING CURB
	EXISTING CURB AND GUTER
	PROPOSED CURB AND GUTTER WITH SIDEWALK
	EXISTING HYDRANT
	HYDRANT, VALVE AND TEE
	EXISTING WATER SUPPLY WELL
	NEW WATER SUPPLY WELL
	FINISHED FLOOR ELEV.
	BASEMENT / CRAWL SPACE ELEV.
	TO BE SERVICED
	APPROX. SEPTIC TANK LOCATION
	TEST PIT LOCATION
	BORE HOLE LOCATION
	EXISTING SIGNAGE
	HYDRO POLE AND GUY WIRE
	SLOPE
0.00	SPOT ELEVATION
	PROPOSED FINISH GRADE
	FLAG POLE
	LAMP POST
	POST
	EXPOSED ROCK
	ON PLAN INDICATES AREA TO BE PAVED WITH 200mm COMPACTED CLASS 'B', 100mm COMPACTED CLASS 'A' GRANULAR, 40mm BASE COURSE ASPHALT AND 40mm SURFACE COURSE ASPHALT, WITH 1.0m WIDE SHOULDERS EACH SIDE.
	ON PLAN INDICATES NEW GRAVEL ROAD WAY WITH 200mm COMPACTED CLASS 'B' AND 100mm COMPACTED CLASS 'A' GRANULAR, WITH 1.0m WIDE SHOULDERS EACH SIDE.
PCS - 76G2549 N 5 223 876.022 E 366 893.781 ELEV. = 21.431m	PROVINCIAL CONTROL SURVEY MONUMENT - (Note: For information on survey control data visit web site - http://www.landgazette.com/free.aspx. Open 'Monument Picker' and enter monument No. or pick a community from the list to obtain monument information

No.	Issue	Date
A	ISSUED FOR APPROVAL	21/JULY/2017
T	ISSUED FOR TENDER	28/JULY/2017
C	ISSUED FOR CONSTRUCTION	08/SEPT/2017
AB	AS-BUILT	02/MAY/2018

No.	Revision	Date

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Conception Bay South, NL | A1W 3A6
T: 1 877 757 7204 | F: 709 782 8941
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AS-BUILT

Professional Seal(s)

REGISTERED PROFESSIONAL ENGINEER
PEG Newfoundland and Labrador
STEVEN FRIZZELL
SIGNATURE
Sept. 8/2017
DATE
NEWFOUNDLAND AND LABRADOR

PROVINCE OF NEWFOUNDLAND AND LABRADOR
PEG Newfoundland and Labrador
PERMIT HOLDER
This Permit Allows
CAP MANAGEMENT SERVICES LTD.
To practice Professional Engineering in Newfoundland and Labrador.
Permit No. as issued by PEGNL N0818 which is valid for the year 2017
Member in Responsible Charge 08253

Drawn By:	G.J.B.R.
Dwg Standards Ckd By:	C.R.M.
Designed By:	C.R.M.
Design Approved By:	S.F.
Scale:	SCALE AS NOTED

Client Title

TOWN OF MARYSTOWN
NEWFOUNDLAND AND LABRADOR

Project. Title

CANNING BRIDGE WATER MAIN REPLACEMENT

Dwg. Title

STANDARD CONSTRUCTION NOTES AND DRAWING LEGEND

Project No. **NL00007**

MAE No. **17-MYCW-18-00123**

Dwg. No. **C3** | Rev. No. **0**

Printed: 2018-04-26 13:24:53 UTC

DataLogger VAULT

McElroy Joint Report

Reference Number 922672

Job Details

Joint Number	1
Joint Time	2018-04-14 11:01:37 GMT
Job	marystown
Operator	ben

Fusion Machine

Machine Name	unit 255
Machine Model	T500 MF
Piston Area	6.01 in²

Pipe Specifications

Pipe Material	PE4710
Pipe Size	12 " IPS
Wall Thickness	DR 11

Pressures

Drag Pressure 51 psi

	Interfacial	Gauge
Bead Up	75 psi	578 psi
Heat Soak	0 psi	51 psi
Fuse	75 psi	578 psi
Cool	0 psi	0 psi

Fusion Specification

Fusion Type	Butt Fusion
Fusion Specification	ASTM F2620
Bead Time	0 seconds
Bead Size	1/4"
Heat/Soak Time	312 seconds
Fuse Time	764 seconds
Open/Close Time	15 seconds
Cool Time	0 seconds

	Minimum	Maximum
Bead Up	472 psi	683 psi
Heat Soak	0 psi	51 psi
Fuse	472 psi	683 psi
Cool	0 psi	0 psi

External Heater Temperatures

	Side A	Side B
One	425 F	431 F
Two	427 F	429 F
Three	433 F	427 F
Four	430 F	435 F

GPS Location

	Latitude	Longitude
2018-04-14 14:36:36 UTC	47°9'36.9"N	55°9'34.8"W

Logged Data Summary

Number of Data Points	144
Total Fusion Time	2024 seconds
Maximum Recorded Pressure	579 psi

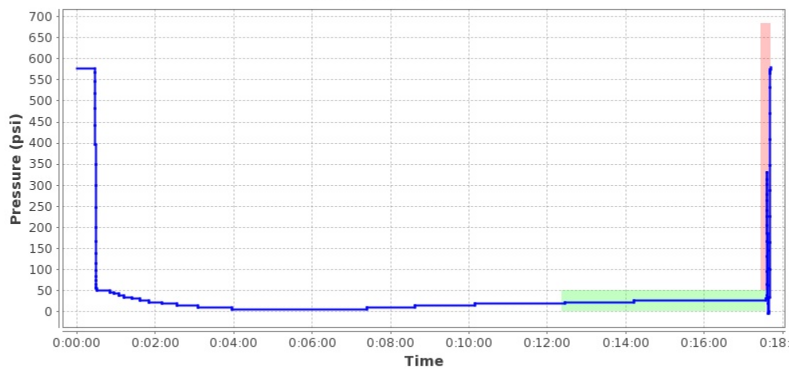
Device Information

DataLogger Serial Number	MDL5-0050
Calibration Date	2017-06-27
Firmware Version	v5.1
Software Version	v1.1.2
Software Product Name	DL5m

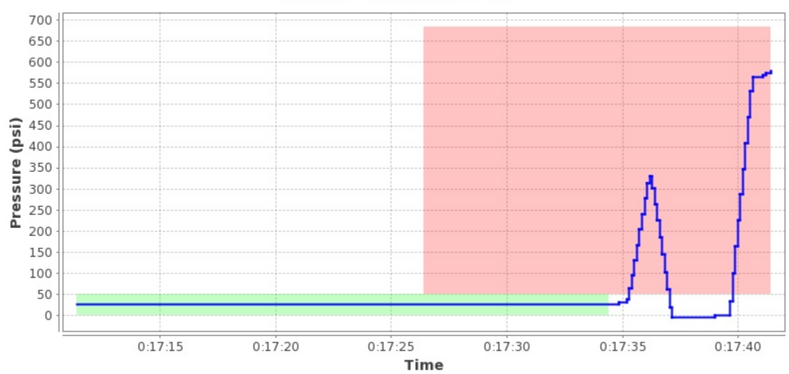
Data Source

File Name \My Documents\Joint Reports\DL5 2018-04-14 11-01-37 Joint 1 Job marystown by ben.DL5
Upload Time 2018-04-26 12:51:32 GMT

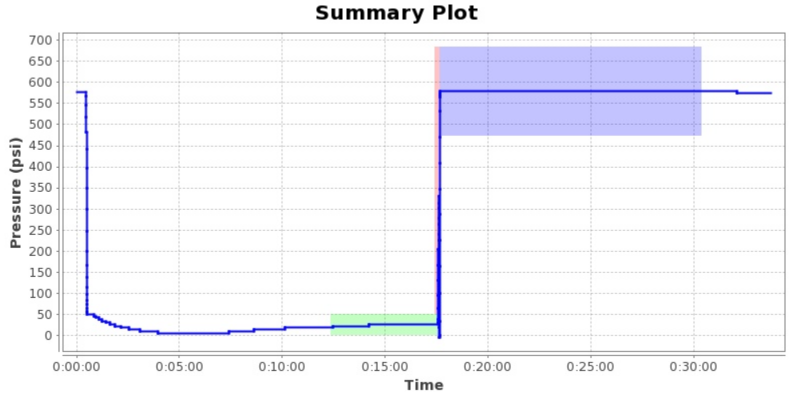
Front-end Plot



Heater Removal Plot



Summary Plot



Notes

APPENDIX B

Inspection Forms

Bridge Inspection Form

1.0 Inventory Data

Structure Name:	Marystown Harbour Bridge
Bridge ID:	2-003
Route:	220, km 0.7
Structure Location:	Marystown, NL
GPS Location:	Lat 47.159420 Long -55.159933
Superstructure Type:	Steel Plate Girder
Substructure Type:	Reinforced Concrete Abutments and Piers
Total Deck Length (m):	119.5
Overall Bridge Width (m):	8.5
Roadway Width (m):	6.2
Total Deck Area (sq. m):	740.9
No. of Lanes:	2
Posted Speed (km/h):	50
Crossing Type:	Over Water

	Span 1	Span 2	Span 3	Span 4	Span 5
Span Lengths (m):	24.384	30.48	24.384	19.812	19.812

2.0 Historical Data

Year Built:	1957
Year of Last Rehab:	2010
Last Inspection:	July 11, 2018

Rehab History (Year/description):

1991	New deck, deck drains, sidewalk, curb, expansion joints, and alumimum rails
2010	Expansion joint replacement and associated work

Considerations / Defects known to the Department (Item/Description):

A	Cracking on abutments and piers
B	Corrosion of bearings
C	Deck concrete delamination
D	Concrete disintegration all around piers
E	Steel girders showing significant areas of rust
F	Girder bracing showing signs of section loss

3.0 Field Inspection Information

Date of Inspection:	September 26 to 30, 2022
Inspectors:	CBCL Limited via Todd Puddicome, P. Eng & Mitchell Warren, EIT
Traffic Control:	Safety First
Access Equipment:	Rope Access (by Acuren) / Dive Inspection by SEA-Force

	Day 1	Day 2	Day 3	Day 4	Day 5
Weather:	Sunny	Sunny	Rain	Sunny	Sunny
Temp (deg C):	22	24	20	15	15

4.0 Overall Structure Recommendations

Work Category:	Major Rehab / Structure Replacement
Timing:	1-5 Year
Est Total Cost:	>$10,000,000
Comments:	Substructure is in poor condition. All piers are deteriorating due to water/ice erosion. Superstructure: Painted coating is in poor condition and underlying steel is corroding

5.00	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Abutment Walls	
Location:	North	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	N/A	
Width (m):	8.1	
Height (m):	3.8	
Count:	1	
Total Quantity (m^2):	30.5	
Limited Inspection?	Yes	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	16.7	1.8	12

Comments:

Very severe delamination with wide cracks present at the following locations:
- along top edge of wall for full length: 7.5m x 0.4m
- in lower left quadrant: 0.5m x 0.5m
- vertically along eastern edge: 0.5m x 0.5m

There is 6m of medium cracking in the lower portion of the wall

Only 2.0m of abutment wall is visible above ground

Approx 80% of surface of beam seat is severely delaminated

	Performance Deficiencies
1	00 - None
2	

	Maintenance Needs	**Timing**
1	08 - Repair of bridge concrete	1 Year
2		

	Recommended Work	**Category**	**Timing**
1	Remove and repair concrete	Rehabilitation	1 - 5 years
2			

5.01	Element Data

Element Data		Sketch (if required):
Element Group:	Abutments	
Element Name:	Abutment Walls	
Location:	South	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced concrete	
Environment:	Severe	
Protection System:	None	
Length (m):	N/A	
Width (m):	8.1	
Height (m):	5.9	
Count:	1	
Total Quantity (m^2):	47.9	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2	0	31.3	2.175	14.5

Comments:

1. Severely delamination present at the following locations:
- vertically along eastern edge full height (0.5m wide)
- Upper left quadrant: 0.3m x 0.3m
- Upper right quadrant: 0.6m x 0.3m
- vertically along western edge 0.5m x 0.4m

2. Medium spalling present at the following locations:
- upper left quadrant: 0.2m x 0.2m
- western edge: 0.2m x 0.3m

3. Light erosion along bottom of wall (600mm wide) full length

4. 15m of narrow to medium cracks dispersed over wall

5. Severe delamination at midway on beam seat to west side (0.6m x 3m)

6. ~ 0.6m of wall is underwater at high tide

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1	08 - Repair bridge concrete	1 year
2	-	

	Recommended Work	Category	Timing
1	Remove and repair concrete	Rehabilitation	1 - 5 years
2			

5.02	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Ballast Walls	
Location:	North	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced Concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	N/A	
Width (m):	8.125	
Height (m):	2.72	
Count:	1	
Total Quantity (m^2):	22.1	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	14.3	1.02	6.8

Comments:

1. Very severe delamination present in the following areas:
- western edge full height 0.7m wide
- 0.7m in from western edge: 1.4m x 0.9m
- eastern edge: 1.7m x 2.4m
- eastern edge: 0.9m x 0.5m

2. Map cracking on 80% of surface: narrow cracks (40%) and medium cracks (40%)

3. Medium map cracking on 40% of surface

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Repair of bridge concrete	1 year
2		

	Recommended Work	**Category**	**Timing**
1			
2			

5.03	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Ballast Wall	
Location:	South Abutment	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced Concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	N/A	
Width (m):	8.125	
Height (m):	1.25	
Count:	1	
Total Quantity (m^2):	10.2	
Limited Inspection?	Yes	

	Unit	Excellent	Good	Fair	Poor
Condition Data	m^2		9.0	0.15	1

Comments:

1. Medium to very severe delamination present at the following locations:
- Eastern edge: 1.0m x 0.4m
- Middle: 0.4m x 0.7m
- Near Girder G2: 0.2m x 0.2m

2. Wide Cracks
- 2m vertical at middle of wall
- 0.7 horizontal at girder G2

3. Original ballast wall (between girders) is covered with a concrete wall from the 1991 rehab

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1	08 - Repair bridge concrete	1 Year
2		

	Recommended Work	Category	Timing
1			
2			

5.04	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Wingwall	
Location:	Northwest	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced Concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	3.9	
Width (m):	N/A	
Height (m):	2.35	
Count:	1	
Total Quantity (m^2):	9.17	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	9.17	0	0

Comments:

1. Narrow to medium map cracking with efflourescence present on 100% of surface

Performance Deficiencies	
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Repair of bridge concrete	2 Years
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

5.05	Element Data

Element Group:	Abutments
Element Name:	Wingwall
Location:	Northeast
Material:	Cast-in-place Concrete
Element Type:	Reinforced Concrete
Environment:	Moderate
Protection System:	None
Length (m):	2.3
Width (m):	N/A
Height (m):	4.5
Count:	1
Total Quantity (m^2):	10.35
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2	0	9.2	0.15	1

Comments:

1. Narrow to medium map cracking with efflourescence is present on 100% of surface

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1	08 - Repair of bridge concrete	1 Year
2	-	

	Recommended Work	Category	Timing
1			
2			

5.06	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Wingwall	
Location:	Southwest	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced Concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	6	
Width (m):	N/A	
Height (m):	4.5	
Count:	1	
Total Quantity (m^2):	27	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		25.39	0.21	1.4

Comments:

1. Very severe delamination with wide cracks (8m long total) is present on edge: 2.5m x 0.4m
2. Cracking: 5m medium cracking, 5m wide cracking, 5m narrow cracking

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Repair bridge concrete	2 Years
2		

	Recommended Work	**Category**	**Timing**
1			
2			

5.07	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Wingwall	
Location:	Southeast	
Material:	Cast-in-place Concrete	
Element Type:	Reinforced Concrete	
Environment:	Moderate	
Protection System:	None	
Length (m):	3	
Width (m):	N/A	
Height (m):	2.85	
Count:	1	
Total Quantity (m^2):	8.55	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	0	0	8.55

Comments:

1. Very severe delamination on 100% surface with narrow map cracking

Performance Deficiencies	
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Repair bridge concrete	2 Years
2		

	Recommended Work	**Category**	**Timing**
1			
2			

5.08	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Bearings	
Location:	North	
Material:	Steel	
Element Type:	Rocker Bearing	
Environment:	Moderate	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (each):	2	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each				2

Comments:

1. Medium corrosion on all components
2. Bearings are potentially seized due to corrosion
3. Nuts on masonry plate anchors have experienced ~95% section loss

	Performance Deficiencies
1	05 - Seized Bearing
2	-

	Maintenance Needs	Timing
1	06 - Bridge Bearing Maintenance	2 Years
2	07 - Repair to Structural Steel	2 Years

	Recommended Work	Category	Timing
1			
2			

5.09	Element Data

Element Group:	Abutments	**Sketch (if required):**
Element Name:	Bearings	
Location:	South	
Material:	Steel	
Element Type:	Non-conventional	
Environment:	Moderate	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (each):	2	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each				2

Comments:

1. Bearings are wheels (from original operable swing span) that are locked into position transverse to the span
2. No signs of differential movement or expansion/contraction issues.

	Performance Deficiencies
1	05 - Seized Bearing
2	-

	Maintenance Needs	**Timing**
1	06 - Bridge Bearing Maintenance	2 Years
2	07 - Repair to Structural Steel	2 Years

	Recommended Work	**Category**	**Timing**
1			
2			

5.10	Element Data

Element Group:	Approaches	**Sketch (if required):**
Element Name:	Wearing Surface	
Location:	North	
Material:	Asphalt	
Element Type:	N/A	
Environment:	Severe	
Protection System:	N/A	
Length (m):	5	
Width (m):	6.2	
Height (m):	0.05	
Count:	1	
Total Quantity (m^2):	31	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2	0	0	28	3

Comments:

1. Depression in asphalt in both lanes (2m x 1m)
2. Uneven ride. Cars slow down when entering bridge. Potential risk of car collision. We witnessed three near-misses
3. Medium crack in asphalt (3m total)

	Performance Deficiencies
1	09 - Rough Riding Surface
2	-

	Maintenance Needs	Timing
1	12 - Bridge Surface Repair	1 Year
2	-	

	Recommended Work	Category	Timing
1			
2			

5.11	Element Data

Element Group:	Approaches	**Sketch (if required):**
Element Name:	Wearing Surface	
Location:	South	
Material:	Asphalt	
Element Type:	N/A	
Environment:	Severe	
Protection System:	N/A	
Length (m):	5	
Width (m):	6.2	
Height (m):	0.05	
Count:	1	
Total Quantity (m^2):	31	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	29.5	0	1.5

Comments:

1. Medium crack in asphalt at deck interface. No sealant
2. Smooth transition. No potholes present

Performance Deficiencies	
1	00 - None
2	-

Maintenance Needs		**Timing**
1	-	
2	-	

Recommended Work		**Category**	**Timing**
1			
2			

5.12	Element Data

Element Group:	Approaches
Element Name:	Barriers
Location:	North
Material:	Steel
Element Type:	Thrie beam
Environment:	Moderate
Protection System:	Galvanized
Length (m):	10
Width (m):	0.3
Height (m):	0.6
Count:	2
Total Quantity (m):	20
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	m		9		1

Comments:

Northeast:
1. Post #5 is split vertically
2. Rail is dented in the burried end span
3. Appears to be fairly new

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.13	Element Data

Element Group:	Approaches	**Sketch (if required):**
Element Name:	Barriers	
Location:	South	
Material:	Steel	
Element Type:	Thrie beam	
Environment:	Moderate	
Protection System:	Galvanized	
Length (m):	10	
Width (m):	0.3	
Height (m):	0.6	
Count:	2	
Total Quantity (m^2):	20	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m		9		1

Comments:

1. Dent in top half of beam at last post on west side
2. Appears to be fairly new

Performance Deficiencies	
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.14	Element Data

Element Group:	Barriers	**Sketch (if required):**
Element Name:	Railing System	
Location:	Full Bridge	
Material:	Aluminum	
Element Type:	Four Rail	
Environment:	Severe	
Protection System:	Paint	
Length (m):	119.5	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (m):	239.0	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m		40	180	20

Comments:

1. East Side:
- all four rails are deflected 50mm near post 27 (from North). Wear markings are present. Suspected vehicular impact
- 2nd rail has continuous minor wear along full length (possible snow plow damage)
- dent 2m long on second rail near post 11
2. West Side:
- third rail down from top has impact damage 0.6m long

3. Railing does not meet TL requirements

	Performance Deficiencies
1	01 - Load Capacity
2	

	Maintenance Needs	Timing
1	-	
2	-	

	Recommended Work	Category	Timing
1	Replace railing	Replacement	1-5 years
2			

5.15	Element Data

Element Group:	Barriers	**Sketch (if required):**
Element Name:	Posts	
Location:	Full Bridge	
Material:	Aluminum	
Element Type:	Posts	
Environment:	Severe	
Protection System:		
Length (m):	N/A	
Width (m):	N/A	
Height (m):	0.914	
Count:	41	
Total Quantity (Each):	41	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each		40		1

Comments:

East Side:
1. Dent on post 27

	Performance Deficiencies
1	01 - Load Capacity
2	

	Maintenance Needs	**Timing**
1	-	
2	-	

	Recommended Work	**Category**	**Timing**
1	Replacing railing	Replacement	1-5 Years
2			

5.16	Element Data

Element Data		Sketch (if required):
Element Group:	Beams	
Element Name:	Girders	
Location:	West	
Material:	Steel	
Element Type:	Plate Girder	
Environment:	Severe	
Protection System:	Paint	
Length (m):	79.9	
Width (m):	0.43	
Height (m):	1.82	
Count:	1	
Total Quantity (m^2):	393.9	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2			183.9	210

Comments:

1. Section loss is present along top and bottom flange
2. Nuts and protruding bolts on splice plates are severely corroded on all plates
3. Rust blistering is present along web surface (10%)
4. Light to medium surface corrosion along all web-flange interfaces
5. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.
6. Stiffener directly over bearing is very severely corroded with 50% section loss at base (one side of beam)

	Performance Deficiencies
1	01 - Load Carrying Capacity
2	-

	Maintenance Needs	Timing
1	07 - Repair structural steel	1 Year
2		

	Recommended Work	Category	Timing
1	Reinforce top & bottom flange	Rehabilitation	1 - 5 years
2	Replace splice plate bolts	Rehabilitation	1 - 5 years

5.17	Element Data

Element Group:	Beams	**Sketch (if required):**
Element Name:	Girders	
Location:	East	
Material:	Steel	
Element Type:	Plate Girder	
Environment:	Severe	
Protection System:	Paint	
Length (m):	79.9	
Width (m):	0.43	
Height (m):	1.82	
Count:	1	
Total Quantity (m^2):	393.9	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2			183.9	210

Comments:

1. Section loss is present along top and bottom flange
2. Nuts and protruding bolts on splice plates are severely corroded on all plates
3. Rust blistering is present along web surface (10%)
4. Ligth to medium surface corrosion along all web-flange interfaces
5. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	01 - Load Carrying Capacity
2	-

	Maintenance Needs	**Timing**
1	07 - Repair structural steel	1 - 5 Years
2		

	Recommended Work	**Category**	**Timing**
1	Reinforce top & bottom flange	Rehabilitation	1 - 5 years
2	Replace splice plate bolts	Rehabilitation	1 - 5 years

5.18	Element Data

Element Group:	Beams	**Sketch (if required):**
Element Name:	Girders	
Location:	Swing Span East	
Material:	Steel	
Element Type:	Tapered Plate Girder	
Environment:	Severe	
Protection System:	Paint	
Length (m):	39.624	
Width (m):	0.46	
Height (m):	1.35	
Count:	1	
Total Quantity (m^2):	179.9	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2			100	80

Comments:

1. Medium to severe corrosion present on 75% of bottom flange with section loss is present. Top flange encased in concrete.
2. Nuts and protruding bolts on splice plates are severely corroded on all plates
3. Rust blistering is present along web surface (5%)
4. Light to medium surface corrosion along all web-flange interfaces
5. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

Performance Deficiencies	
1	01 - Load Carrying Capacity
2	-

	Maintenance Needs	**Timing**
1	07 - Repair structural steel	1 - 5 Years
2		

	Recommended Work	**Category**	**Timing**
1	Reinforce bottom flange	Rehabilitation	1 - 5 years
2	Replace bolts in splice plates	Rehabilitation	1 - 5 years

5.19	Element Data

Element Group:	Beams	**Sketch (if required):**
Element Name:	Girders	
Location:	Swing Span West	
Material:	Steel	
Element Type:	Tapered Plate Girder	
Environment:	Severe	
Protection System:	Paint	
Length (m):	39.624	
Width (m):	0.46	
Height (m):	1.35	
Count:	1	
Total Quantity (m^2):	179.9	
Limited Inspection?	Yes	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2			80	100

Comments:

1. Corrosion present on 75% of bottom flange with section loss. Top flange encased in concrete
2. Nuts and protruding bolts on splice plates are severely corroded on all plates
3. Rust blistering is present along web surface (5%)
4. Light to medium surface corrosion along all web-flange interfaces
5. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	01 - Load Carrying Capacity
2	-

	Maintenance Needs	**Timing**
1	07 - Repair structural steel	1 - 5 Years
2		

	Recommended Work	**Category**	**Timing**
1	Reinforce bottom flange	Rehabilitation	1 - 5 years
2	Replace bolts in splice plates	Rehabilitation	1 - 5 years

5.20	Element Data

Element Group:	Beams	**Sketch (if required):**
Element Name:	Floor Beams	
Location:	Span 1, 2, &3	
Material:	Steel	
Element Type:	W410x67	
Environment:	Severe	
Protection System:	Paint	
Length (m):	7.982	
Width (m):	0.179	
Height (m):	0.41	
Count:	27	
Total Quantity (m^2):	292.5	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2		72.5	220	

Comments:

1. Corrosion is present on 50% of bottom flanges. No appreciable section loss has occurred yet
2. Rust blistering is present along web surface (5%)
3. Light to medium surface corrosion along all web-flange interfaces
4. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

Performance Deficiencies	
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.21	Element Data

Element Group:	Beams
Element Name:	Floor Beams
Location:	Swing Span 1 & 2
Material:	Steel
Element Type:	W410x54
Environment:	Severe
Protection System:	Paint
Length (m):	7.982
Width (m):	0.175
Height (m):	0.41
Count:	10
Total Quantity (m^2):	107.4
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2		26.4	81	

Comments:

1. Corrosion is present on 50% of bottom flanges. No appreciable section loss has occurred yet
2. Rust blistering is present along web surface (5%)
3. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.22	Element Data

Element Group:	Beams	**Sketch (if required):**
Element Name:	Floor Stringer	
Location:	Swing Span 1 & 2	
Material:	Steel	
Element Type:	W360x51	
Environment:	Severe	
Protection System:	Paint	
Length (m):	4.8768	
Width (m):	0.17	
Height (m):	0.36	
Count:	16	
Total Quantity (Each):	16.0	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	16		4	12	

Comments:

1. Corrosion is present on 50% of bottom flanges. No appreciable section loss has occurred yet
2. Rust blistering is present along web surface (5%)
3. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1			
2			

5.23	Element Data

Element Group:	Beams
Element Name:	Diaphragms
Location:	Cont Span
Material:	Steel
Element Type:	L100x100x8
Environment:	Severe
Protection System:	Paint
Length (m):	8.8
Width (m):	0.1
Height (m):	0.1
Count:	10
Total Quantity (Each):	10.0
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each				10

Comments:

1. Very severe corrosion on all members
2. Remaining strength of members is suspect. Rope Access Techs would not tie off to it
3. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

Performance Deficiencies	
1	01 - Load Carrying Capacity
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Remove and replace diaphragm members	Replacement	1 - 5 years
2			

5.24	Element Data

Element Group:	Bracing
Element Name:	Bracing
Location:	Span 1, 2, 3
Material:	Steel
Element Type:	Sidewalk Diagonal
Environment:	Severe
Protection System:	Paint
Length (m):	2.1
Width (m):	0.075
Height (m):	0.15
Count:	54
Total Quantity (Each):	54.0
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each		54		

Comments:

1. Light corrosion on members. No appreciable section loss present

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.25	Element Data

Element Data		Sketch (if required):
Element Group:	Bracing	
Element Name:	Bracing	
Location:	Swing Span 1 & 2	
Material:	Steel	
Element Type:	Sidewalk Diagonal	
Environment:	Severe	
Protection System:	Paint	
Length (m):	2.1	
Width (m):	0.075	
Height (m):	0.15	
Count:	10	
Total Quantity (Each):	20.0	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each		20		

Comments:

1. Light corrosion on members. No appreciable section loss present

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1			
2			

5.26	Element Data

Element Group:	Bracing	**Sketch (if required):**
Element Name:	Bracing	
Location:	Span 1, 2, 3	
Material:	Steel	
Element Type:	Hoizontal Diagonal	
Environment:	Severe	
Protection System:	Paint	
Length (m):	11.3	
Width (m):	0.1	
Height (m):	0.1	
Count:	14	
Total Quantity (Each):	14.0	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each				14

Comments:

1. Very severe corrosion on all members
2. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	01 - Load Capacity
2	-

	Maintenance Needs	Timing
1	07 - Structural Steel Repair	1 Year
2		

	Recommended Work	Category	Timing
1			
2			

5.27	Element Data

Element Group:	Bracing	**Sketch (if required):**
Element Name:	Bracing	
Location:	Swing Span 1 & 2	
Material:	Steel	
Element Type:	Horizontal Diagonal	
Environment:	Severe	
Protection System:	Paint	
Length (m):	11.328	
Width (m):	0.125	
Height (m):	0.09	
Count:	8	
Total Quantity (Each):	8.0	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each			1	7

Comments:

1. Very severe corrosion on all members
2. Tacten has performed a visual inspection and UT measurements. Results are available in their independent report.

	Performance Deficiencies
1	01 - Load Capacity
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1	Remove and replace all members	Replacement	1 - 5 years
2			

5.28	Element Data

Element Group:	Coatings
Element Name:	Structural Steel
Location:	Span 1, 2 & 3
Material:	Steel
Element Type:	Girder
Environment:	Severe
Protection System:	Paint
Length (m):	N/A
Width (m):	N/A
Height (m):	N/A
Count:	N/A
Total Quantity (m^2):	1366.0
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2			273	1093

Comments:

1. Coating is in very poor condition. Examples of failure include undercutting, peeling, pinholing, rust spotting, and rust staining.
2. Existing paint may contain hazardous materials. Samples have been extracted for further testing.
3. Rust condition 4
4. Tacten performed dry film thickness measurements. Results availble in their coatings report.

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Sand blast and re-coat	Rehabilitation	1-5 years
2			

5.29	Element Data

Element Group:	Coatings	**Sketch (if required):**
Element Name:	Structural Steel	
Location:	Swing Span 1 & 2	
Material:	Steel	
Element Type:	Girder	
Environment:	Severe	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	N/A	
Total Quantity (m^2):	583.0	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2			146	437

Comments:

1. Coating is in very poor condition. Examples of failure include undercutting, peeling, pinholing, rust spotting, and rust staining.
2. Existing paint may contain hazardous materials. Samples have been extracted for further testing.
3. Rust condition 4
4. Tacten performed dry film thickness measurements. Results availble in their coatings report.

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1	Sand blast and re-coat	Rehabilitation	1-5 years
2			

5.30	Element Data

Element Group:	Deck	**Sketch (if required):**
Element Name:	Deck Top	
Location:	Span 1, 2, 3	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:	N/A	
Length (m):	79.9	
Width (m):	8.5	
Height (m):	0.18	
Count:	N/A	
Total Quantity (m^2):	679.15	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2	0	670	1.2	7.8

Comments:

1. Surface wear present on 100% of deck surface. Aggregate is exposed
2. Wear has caused light rutting in wheel tracks of both travel lanes (6mm deep)
3. Medium to severe spalling is present in 11 areas totalling 6 m^2
4. Medium to severe delamination is present in 10 areas totalling 2 m^2

	Performance Deficiencies
1	00 - None
2	

	Maintenance Needs	**Timing**
1	08 - Repair of bridge concrete	1 Year
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

5.31	Element Data

Element Group:	Deck	Sketch (if required):
Element Name:	Deck Top	
Location:	Swing Span 1 & 2	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:	N/A	
Length (m):	40.2	
Width (m):	8.5	
Height (m):	0.18	
Count:	N/A	
Total Quantity (m^2):	341.7	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2	0	337	0.6	3.9

Comments:

1. Surface wear present on 100% of deck surface. Aggregate is exposed
2. Wear has caused light rutting in wheel tracks of both travel lanes (6mm deep)
3. Medium to severe spalling is present in 11 areas totalling 3.5 m^2
4. Medium to severe delamination is present in 10 areas totalling 0.5 m^2

	Performance Deficiencies
1	00 - None
2	

	Maintenance Needs	Timing
1	08 - Repair of bridge concrete	1 Year
2	-	

	Recommended Work	Category	Timing
1			
2			

5.32	Element Data

Element Group:	Deck	**Sketch (if required):**
Element Name:	Soffit	
Location:	Span 1, 2, 3	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:	N/A	
Length (m):	164	
Width (m):	5.3	
Height (m):	0.1	
Count:	N/A	
Total Quantity (m^2):	869.2	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		869.2		

Comments:

1. No delamination or spalling observed

Performance Deficiencies	
1	00 - None
2	

Maintenance Needs		**Timing**
1	-	
2	-	

Recommended Work		**Category**	**Timing**
1			
2			

5.33	Element Data

Element Group:	Deck	**Sketch (if required):**
Element Name:	Soffit	
Location:	Swing Span 1 & 2	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:		
Length (m):	164	
Width (m):	5.3	
Height (m):	0.18	
Count:	N/A	
Total Quantity (m^2):	869.2	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		868	0.15	1

Comments:

1. Severe spalling present (with exposed rebar) near the south abutment in the following areas:
- near backwall on east side between girder and stringer: 0.5m x 0.5m
- between stringers: 0.25m x 1.3m
- near east stringer: 0.5m x 0.2m
- between west stringer and west girder: 0.5m x 0.5m

	Performance Deficiencies
1	00 - None
2	

	Maintenance Needs	**Timing**
1	08 - Repair of bridge concrete	1 Year
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

5.34	Element Data

Element Group:	Deck	**Sketch (if required):**
Element Name:	Drainage System	
Location:	Full Bridge	
Material:	Steel	
Element Type:	Deck Drain	
Environment:	Severe	
Protection System:		
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	16	
Total Quantity (Each):	16	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each		8		8

Comments:

1. 50% of the deck drains are full of debris

Performance Deficiencies	
1	11 - Deck Drainage
2	

	Maintenance Needs	**Timing**
1	02 - Bridge Cleaning	1 Year
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

5.35	Element Data

Element Group:	Joints	**Sketch (if required):**
Element Name:	Armoring	
Location:	North & South Abutment	
Material:	Steel	
Element Type:		
Environment:	Severe	
Protection System:	None	
Length (m):	6.2	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (m):	12.4	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m		12		

Comments:

1. Armoring is exposed. Light wear.

Performance Deficiencies	
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	02 - Bridge cleaning	2 years
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

5.36	Element Data

Element Group:	Joints	**Sketch (if required):**
Element Name:	Seals and Sealants	
Location:	North & South Abutment	
Material:		
Element Type:		
Environment:	Severe	
Protection System:	None	
Length (m):	5.2	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (each):	2	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each		2		

Comments:

North Abutment
1. Seal appears to have come loose in the Northbound lane (0.2m long) from angle
2. Minor debris buildup in joint
3. No noticeable differential movement

South Abutment
1. Minor debris buildup in joint
2. No noticeable differential movement

Performance Deficiencies	
1	00 - none
2	-

	Maintenance Needs	Timing
1		
2	-	

	Recommended Work	Category	Timing
1			
2			

5.37	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Shaft	
Location:	Pier 1	
Material:	Cast-in-place Concrete	
Element Type:	Wall	
Environment:	Severe	
Protection System:	None	
Length (m):	7.9	
Width (m):	1.8	
Height (m):	10.9	
Count:	1	
Total Quantity (m^2):	212.6	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2		77.6	85	50

Comments:

1. Very severe erosion on all four sides of pier shaft. Aggregate exposed
2. Severe loss of section at tidal zone on upstream and downstream edges.
3. Very severe erosion at nosing. Original angled profile turned to a rounded section.
Pier profile has 'hour-glass' shape due to erosion.
4. Very severe delamination present on all concrete remaining on nosing
5. Wide vertical crack in middle of shaft full height
6. Undermining present on SW corner (0.2m x 0.35m).
7. East and west nosing is good condition underwater. Spalling/erosion occurs in the tidal zone.
8. Heavy marine growth (6" deep) over all concrete below low tide

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Concrete to be scaled and encapsulated	Rehabilitation	1 to 5 years
2			

5.38	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Shaft	
Location:	Pier 2	
Material:	Cast-in-place Concrete	
Element Type:	Wall	
Environment:	Severe	
Protection System:	None	
Length (m):	7.9	
Width (m):	1.8	
Height (m):	9.6	
Count:	1	
Total Quantity (m^2):	187.2	
Limited Inspection?	Yes	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		75.2	75	37

Comments:

1. Very severe erosion on all four sides of pier shaft. Aggregate exposed
2. Severe loss of section at tidal zone on upstream and downstream edges.
3. Very severe erosion at nosing. Original angled profile turned to a rounded section.
4. Very severe delamination present on all concrete remaining on nosing

Pier profile has 'hour-glass' shape due to erosion.

5. Wide vertical crack in middle of shaft full height
6. East and west nosing is in good condition underwater. Spalling/erosion occurs in the tidal zone.
7. Severe spalling at waterline on south face: 0.25m deep x 0.8m long x 0.2m wide
8. Heavy marine growth (6" deep) over all concrete below low tide

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Repair bridge concrete	2 years
2		

	Recommended Work	**Category**	**Timing**
1	Concrete to be scaled and encapsulated	Rehabilitation	1 to 5 years
2			

5.39	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Shaft	
Location:	Pier 3	
Material:	Cast-in-place Concrete	
Element Type:	Wall	
Environment:	Severe	
Protection System:	None	
Length (m):	7.9	
Width (m):	1.8	
Height (m):	10.5	
Count:	1	
Total Quantity (m^2):	204.8	
Limited Inspection?	Yes	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		72.8	100	32

Comments:

1. Very severe erosion on all four sides of pier shaft. Aggregate exposed
2. Severe loss of section at tidal zone on upstream and downstream edges.
3. Nosing has eroded to a rounded section. Pier profile has 'hour-glassed' due to erosion.
4. Very severe delamination present on all concrete remaining on nosing
5. Wide vertical crack in middle of shaft full height.
- Crack is 5mm at bottom, 10mm at midheight, 20mm at 3/4 height, 10mm full height
6. East and west nosing is good condition underwater. Spalling/erosion occurs in the tidal zone.
7. Heavy marine growth (6" deep) over all concrete below low tide

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1	Concrete to be scaled and encapsulated	Rehabilitation	1 to 5 years
2			

5.40	Element Data

Element Group:	Piers
Element Name:	Shaft
Location:	Pivot Pier
Material:	Cast-in-place Concrete
Element Type:	Wall
Environment:	Severe
Protection System:	None
Length (m):	7.3
Width (m):	7.3
Height (m):	16.3
Count:	1
Total Quantity (m^2):	250.0
Limited Inspection?	Yes

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2		12	150	88

Comments:

1. Very severe delamination present on 50% of all facets of inclined soffit
2. Very severe delamination present on 50% of lower shaft wall
3. Narrow to medium map cracking with effluorescence is present on 75% of surface
3. Very severe erosion on all sides at the waterline.
4. Undermining present along North face of swing span footing. See dive report for measurements.
5. Heavy marine growth (6" deep) over all concrete

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Concrete to be scaled and encapsulated	Rehabilitation	1 to 5 years
2			

5.41	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Bearings	
Location:	P1	
Material:	Steel	
Element Type:	Roller	
Environment:	Severe	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (m^2):	2	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each				2

Comments:

Very severe corrosion present on both bearings.
Do not appear to be able to rotate based on the corrosion

	Performance Deficiencies
1	05 - Seized Bearing
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1	Replace Bearing	Rehabilitation	1 - 5 Years
2			

5.42	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Bearings	
Location:	P2	
Material:	Steel	
Element Type:	Fixed	
Environment:	Severe	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (each):	2	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	Each				

Comments:

Very severe corrosion present on both bearings.
Do not appear to be able to rotate based on the corrosion

	Performance Deficiencies
1	05 - Seized Bearing
2	-

	Maintenance Needs	**Timing**
1		
2		

	Recommended Work	**Category**	**Timing**
1	Replace Bearing	Rehabilitation	1 - 5 Years
2			

5.43	Element Data

Element Group:	Piers
Element Name:	Bearings
Location:	P3
Material:	Steel
Element Type:	Rocker
Environment:	Severe
Protection System:	Paint
Length (m):	N/A
Width (m):	N/A
Height (m):	N/A
Count:	4
Total Quantity (m^2):	4
Limited Inspection?	No

Sketch (if required):

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each				4

Comments:

Very severe corrosion present on both bearings.
Do not appear to be able to rotate based on the corrosion

Performance Deficiencies	
1	05 - Seized Bearing
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Replace Bearing	Rehabilitation	1 - 5 Years
2			

5.44	Element Data

Element Group:	Piers	**Sketch (if required):**
Element Name:	Bearings	
Location:	Pivot Pier	
Material:	Steel	
Element Type:	Fixed	
Environment:	Severe	
Protection System:	Paint	
Length (m):	N/A	
Width (m):	N/A	
Height (m):	N/A	
Count:	2	
Total Quantity (each):	2	
Limited Inspection?	Yes	

Condition Data	Unit	Excellent	Good	Fair	Poor
	Each				2

Comments:

Bearing is the original rotating turret mechanism.
Difficult to determine how the bearing is anchored due to the large mass of steel

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1		
2		

	Recommended Work	Category	Timing
1	Replace Bearing	Rehabilitation	1 - 5 Years
2			

5.45	Element Data

Element Group:	Sidewalks/curbs	**Sketch (if required):**
Element Name:	Sidewalk	
Location:	Full Bridge	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:		
Length (m):	119.4816	
Width (m):	1.8	
Height (m):	0.25	
Count:	1	
Total Quantity (m^2):	244.9	
Limited Inspection?	No	

Condition Data	Unit	Excellent	Good	Fair	Poor
	m^2		241.5	0.45	3

Comments:

1. Narrow crack along interior edge (top surface) 10m long in middle of swing span
2. Light spalling on vertical face sidewalk at expansion joint (0.2m x 0.2m)
3. Sidewalk transition from swing span to continuous span is level

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	Timing
1	08 - Concrete Repair	2 Years
2	-	

	Recommended Work	Category	Timing
1			
2			

5.46	Element Data

Element Group:	Sidewalks/curbs	**Sketch (if required):**
Element Name:	Curbs	
Location:	Full Bridge	
Material:	Concrete	
Element Type:		
Environment:	Severe	
Protection System:		
Length (m):	119.5	
Width (m):	0.5	
Height (m):	0.25	
Count:	1	
Total Quantity (m^2):	89.6	
Limited Inspection?	No	

Condition Data	**Unit**	**Excellent**	**Good**	**Fair**	**Poor**
	m^2		88.5	0.15	1

Comments:

1. Medium spalling in curb on south side near the innerface face of the end block (1.5m x 0.2m)
2. Curb on swing span is ~25mm higher than continuous span curb
3. Medium spalling at expansion joint on Northwest vertical face of curb: 0.6m x 0.4m

	Performance Deficiencies
1	00 - None
2	-

	Maintenance Needs	**Timing**
1	08 - Concrete Repair	2 Years
2	-	

	Recommended Work	**Category**	**Timing**
1			
2			

APPENDIX C

CBCL Inspection Photos

Marystown (Canning) Bridge
Photo Record

Photo ID 668 Location South

Element Group Joints

Element Type Armoring

2022-09-29 8:40:48 AM

Photo ID 669 Location South

Element Group Joints

Element Type Armoring

2022-09-29 8:41:06 AM

Photo ID 670 Location South

Element Group Joints

Element Type Armoring

2022-09-29 8:41:21 AM

Photo ID 695 Location North

Element Group Joints

Element Type Armoring

2022-09-29 9:00:25 AM

Looking South

Marystown (Canning) Bridge
Photo Record

Photo ID 700 Location Northeast

Element Group Abutments

Element Type Wingwalls

2022-09-29 1:36:27 PM

Northeast wingwall

Photo ID 702 Location North

Element Group Abutments

Element Type Abutment Walls

2022-09-29 1:38:57 PM

North abutment wall, core 20 location.

Photo ID 718 Location Northwest

Element Group Abutments

Element Type Wingwalls

2022-09-29 2:03:56 PM

Northwest wingwall and core 17 location

Photo ID 725 Location North

Element Group

Element Type

2022-09-29 2:12:39 PM

Deck soffit looking South at pier 1 in front of North abutment.

Marystown (Canning) Bridge Photo Record

Photo ID	730	Location	Northeast
Element Group	Approaches		
Element Type	Barriers		

2022-09-29 2:18:20 PM

Undermining on Northeast corner extension for sidewalk and end block.

Photo ID	741	Location	North Abutment
Element Group	Abutments		
Element Type	Bearings		

2022-09-29 2:59:19 PM

Northeast bearing.

Photo ID	749	Location	North Abutment
Element Group	Abutments		
Element Type	Bearings		

2022-09-29 3:47:51 PM

Northwest bearing looking East.

Photo ID	784	Location	Bridge Deck
Element Group			
Element Type			

2022-09-29 4:27:46 PM

Deck core 3 and defects post 21

Photo ID 785 Location Bridge Deck

Element Group

Element Type

2022-09-29 4:28:21 PM

Typical Delamination. South end of continuous span looking South.

Photo ID 786 Location Bridge Deck

Element Group

Element Type

2022-09-29 4:28:56 PM

Typical delamination. South end of continuous span looking North .

Photo ID 803 Location South

Element Group

Element Type

2022-09-29 4:56:29 PM

MPI 4 location on swing span east girder looking north

Photo ID 814 Location South Abutment

Element Group Abutments

Element Type Abutment Walls

2022-09-29 5:16:34 PM

South abutment front face

Photo ID 821 Location Southwest

Element Group Abutments

Element Type Wingwalls

2022-09-29 5:21:23 PM

Southwest wingwall

Photo ID 840 Location South

Element Group

Element Type

2022-09-30 8:33:14 AM

Deck soffit middle looking north

Photo ID 845 Location Southeast

Element Group

Element Type

2022-09-30 8:38:11 AM

East girder swing span looking west

Photo ID 864 Location Southwest

Element Group Abutments

Element Type Bearings

2022-09-30 8:50:49 AM

Southwest bearing

Marystown (Canning) Bridge
Photo Record

Photo ID 877 Location Southeast

Element Group Abutments

Element Type Bearings

2022-09-30 9:01:32 AM

Southeast bearing

Photo ID 889 Location South

Element Group

Element Type

2022-09-30 9:09:53 AM

Slab deterioration between east girder and stringer

Photo ID 918 Location North

Element Group Beams

Element Type Girders

2022-09-30 10:03:56 AM

East girder

Photo ID 921 Location North

Element Group Beams

Element Type Girders

2022-09-30 10:05:31 AM

Bottom flange east girder looking south. Typical corrosion.

Photo ID 922 Location North

Element Group Beams

Element Type Girders

2022-09-30 10:06:04 AM

East girder looking east

Photo ID 934 Location North

Element Group Beams

Element Type Girders

2022-09-30 10:13:32 AM

West girder bottom flange looking north. Typical Corrosion

Photo ID 957 Location Pier 1

Element Group Piers

Element Type Shaft

2022-09-30 10:28:58 AM

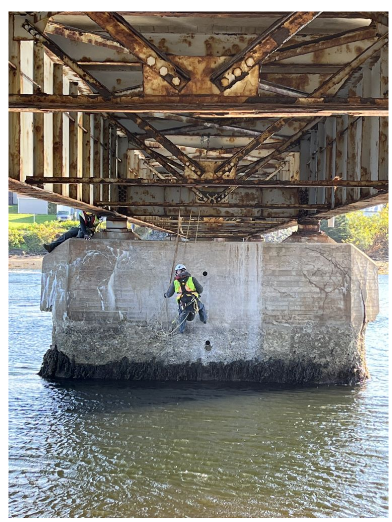
Erosion Pier 1

Photo ID 991 Location Pier 1

Element Group Piers

Element Type Shaft

2022-09-30 4:15:47 PM



Erosion on Pier 1

Marystown (Canning) Bridge
Photo Record

Photo ID 997 Location North
Element Group Beams
Element Type Girders
2022-09-30 4:26:56 PM

West girder splice northwest. Typical corrosion.

Photo ID 1005 Location North
Element Group Piers
Element Type Shaft
2022-09-30 4:34:27 PM

Erosion on piers 1 and 2

Photo ID 1008 Location East Side
Element Group
Element Type
2022-09-30 5:10:46 PM

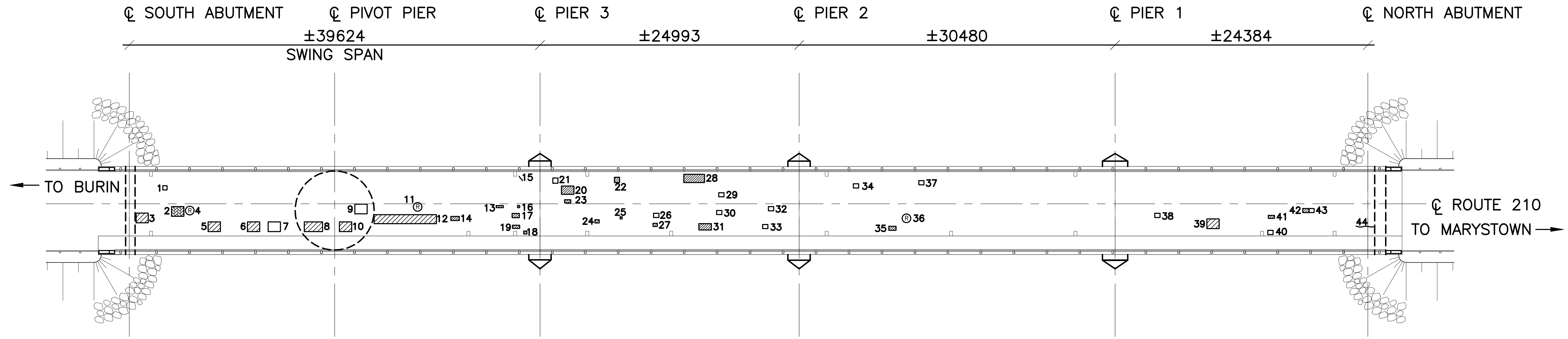
Looking north, Erosion on Piers 1, 2 and 3

Photo ID 1009 Location South
Element Group Piers
Element Type Shaft
2022-09-30 5:11:17 PM

Swing pier

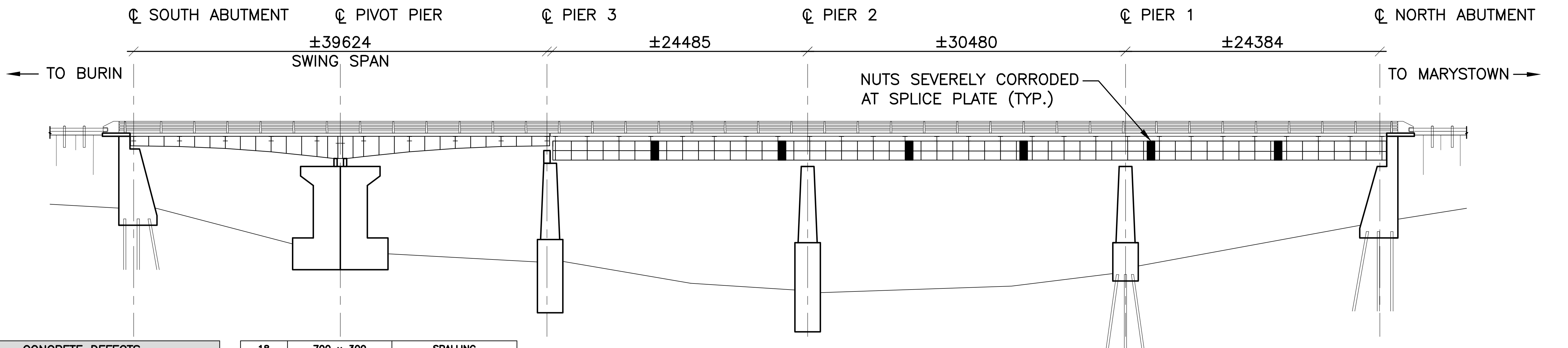
APPENDIX D

Deficiency Sketches



EXISTING PLAN

SCALE 1 : 400



EXISTING ELEVATION

SCALE 1 : 400

CONCRETE DEFECTS

MARK	SIZE	DESCRIPTION
1	400 x 400	DELAMINATION
2	400 x 400	SPALLING
3	800 x 600	SPALLING
4	–	EXPOSED REBAR
5	600 x 400	SPALLING
6	500 x 400	SPALLING
7	300 x 300	DELAMINATION
8	500 x 400	SPALLING
9	600 x 400	SPALLING
10	400 x 400	SPALLING
11	–	EXPOSED REBAR
12	2000 x 400	SPALLING
13	700 x 200	SPALLING
14	800 x 400	SPALLING
15	600	CRACK
16	200 x 200	SPALLING
17	700 x 400	SPALLING
18	700 x 300	SPALLING
19	300 x 300	SPALLING
20	1200 x 600	SPALLING
21	500 x 500	DELAMINATION
22	500 x 200	SPALLING
23	600 x 300	SPALLING
24	400 x 300	SPALLING
25	200 x 200	SPALLING
26	500 x 400	DELAMINATION
27	400 x 300	SPALLING
28	2000 x 1000	SPALLING
29	400 x 400	DELAMINATION
30	400 x 400	DELAMINATION
31	1200 x 600	SPALLING
32	500 x 400	DELAMINATION
33	500 x 400	DELAMINATION
34	500 x 500	DELAMINATION
35	2000 x 500	SPALLING
36	–	EXPOSED REBAR
37	300 x 300	DELAMINATION
38	500 x 300	DELAMINATION
39	1000 x 500	SPALLING
40	500 x 300	DELAMINATION
41	400 x 400	SPALLING
42	400 x 400	SPALLING
43	400 x 400	DELAMINATION
44	2000	CRACK

NOTE:
1. EXPOSED AGGREGATE ON 100% OF DECK SURFACE.
2. RUTTING IN WHEEL PATHS (6mm DEEP)

No.	Description
A	ISSUED FOR DRAFT REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:400	T.P.	J.N.	T.P.	C.J.	223049.10	

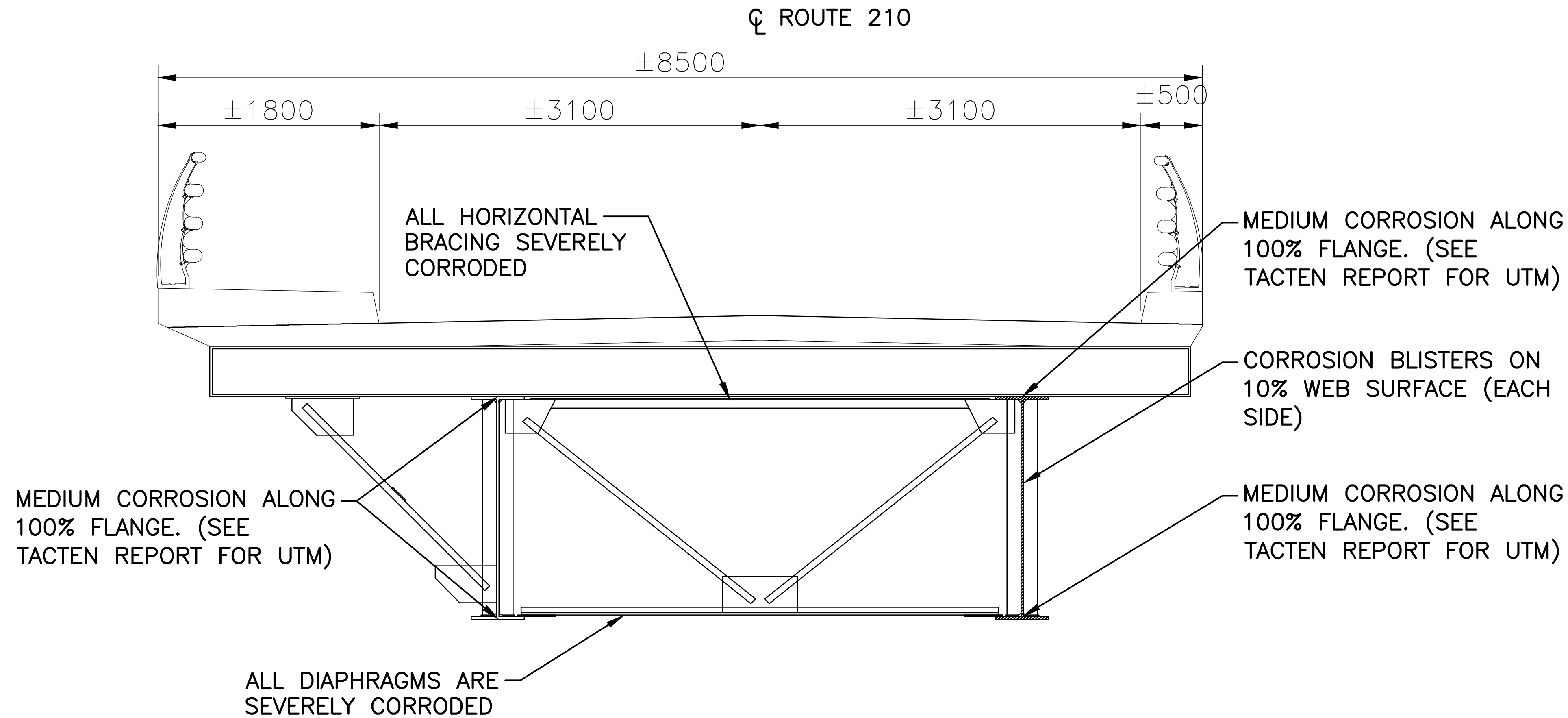
CBCL

MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

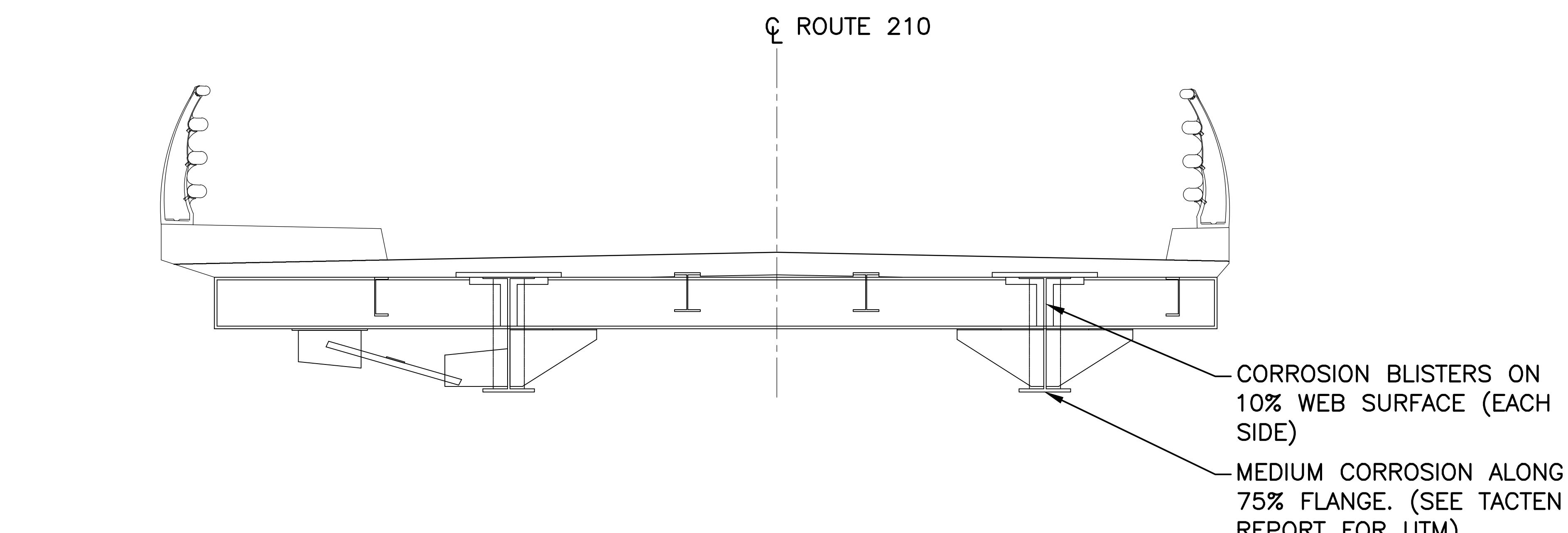
GENERAL ARRANGEMENT

Drawing

S-001



TYPICAL DECK SECTION – CONT. SPAN



TYPICAL DECK SECTION – SWING SPAN

No.	Description
A	ISSUED FOR DRAFT REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:50	T.P.	J.N.	T.P.	C.J.	223049.10	

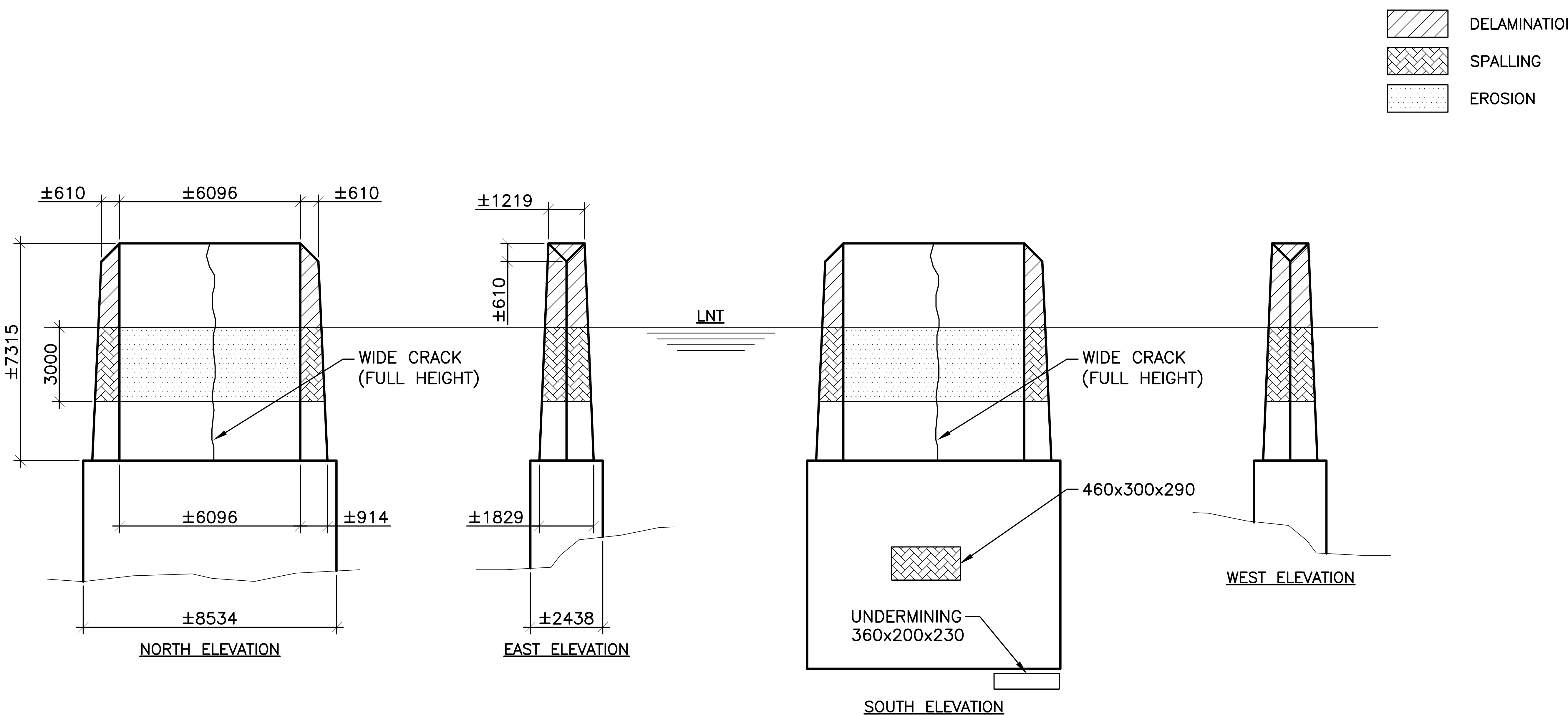
CBCL

MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

GENERAL ARRANGEMENT - CROSS SECTIONS

Drawing

S-002



Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:150	T.P.	J.N.	T.P.	C.J.	223049.10	

No.	Description
A	ISSUED FOR DRAFT REPORT

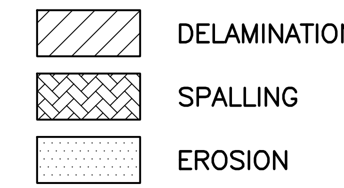
CBCL

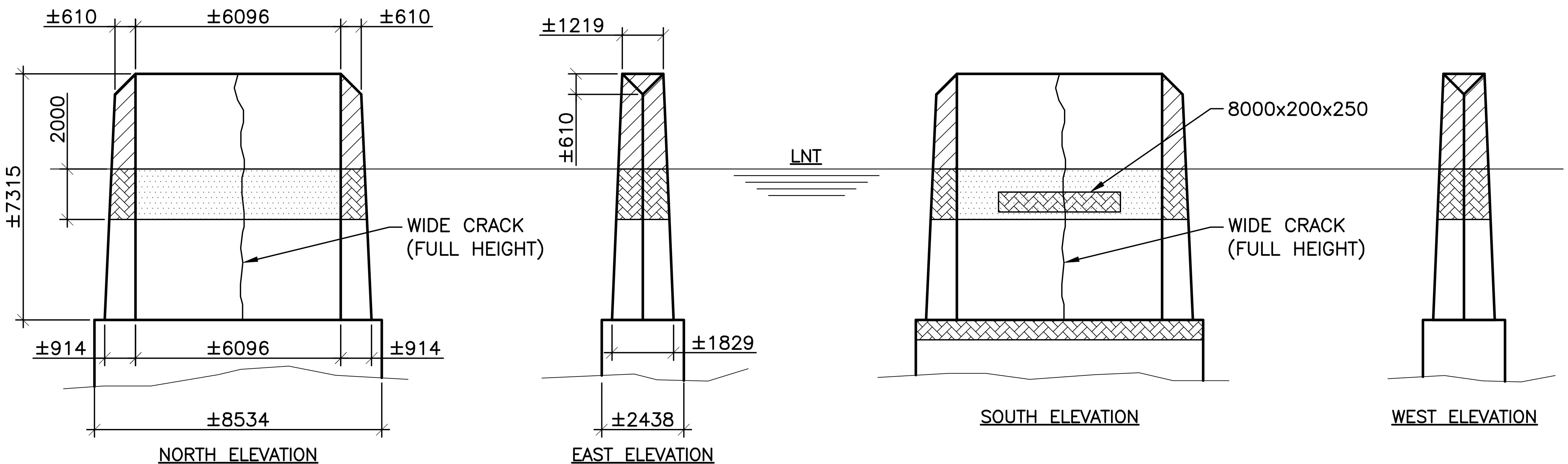
MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

PIER 1 - ELEVATIONS

Drawing

S-100





Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:150	T.P.	J.N.	T.P.	C.J.	223049.10	

No.	Description
A	ISSUED FOR DRAFT REPORT

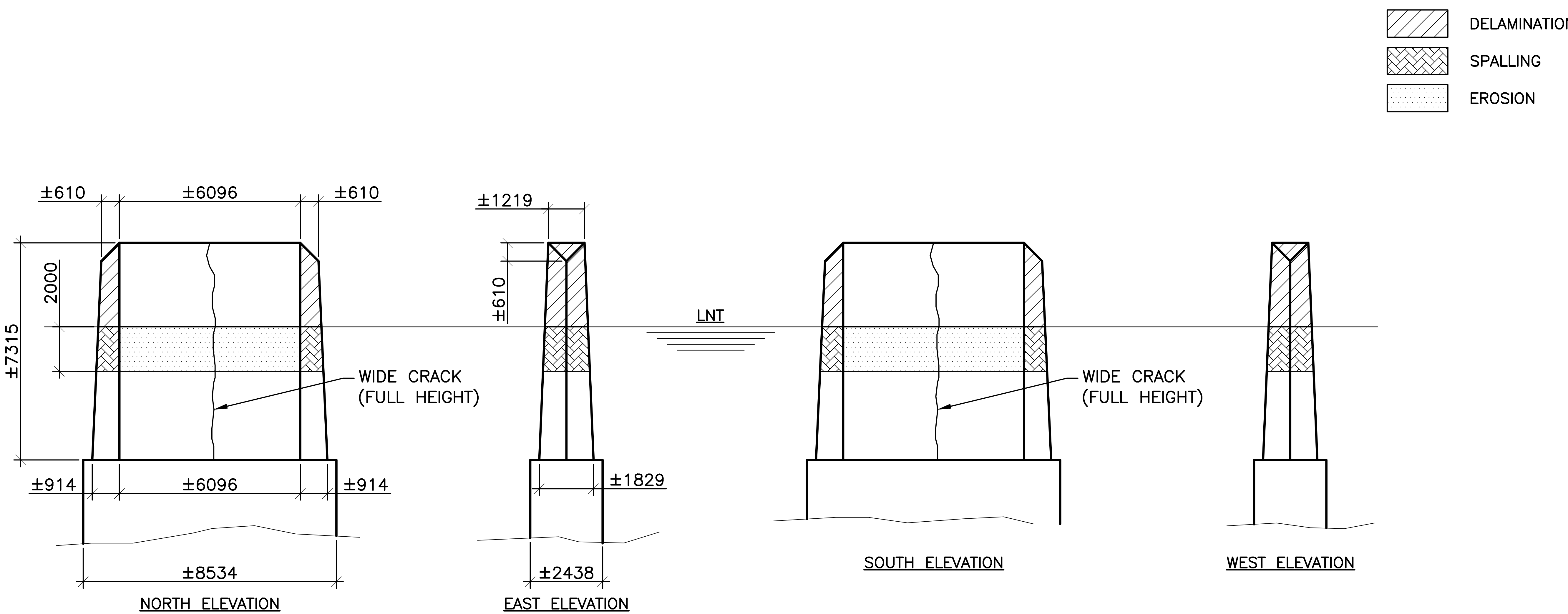
CBCL

MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

PIER 2 - ELEVATIONS

Drawing

S-101



No.	Description
A	ISSUED FOR DRAFT REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:150	T.P.	J.N.	T.P.	C.J.	223049.10	

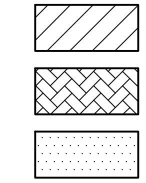
CBCL

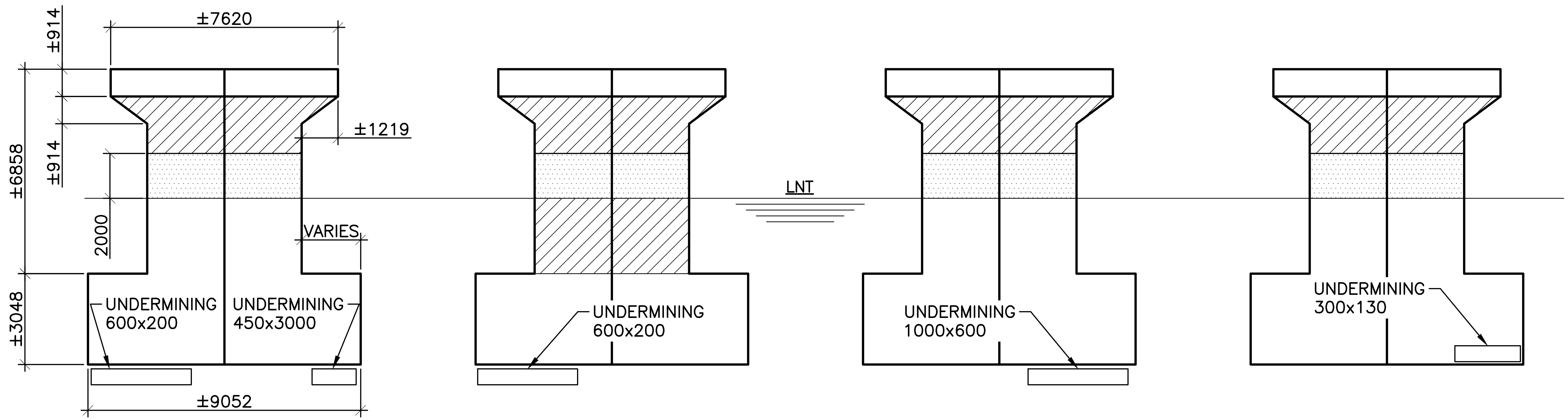
MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

PIER 3 - ELEVATIONS

Drawing

S-103



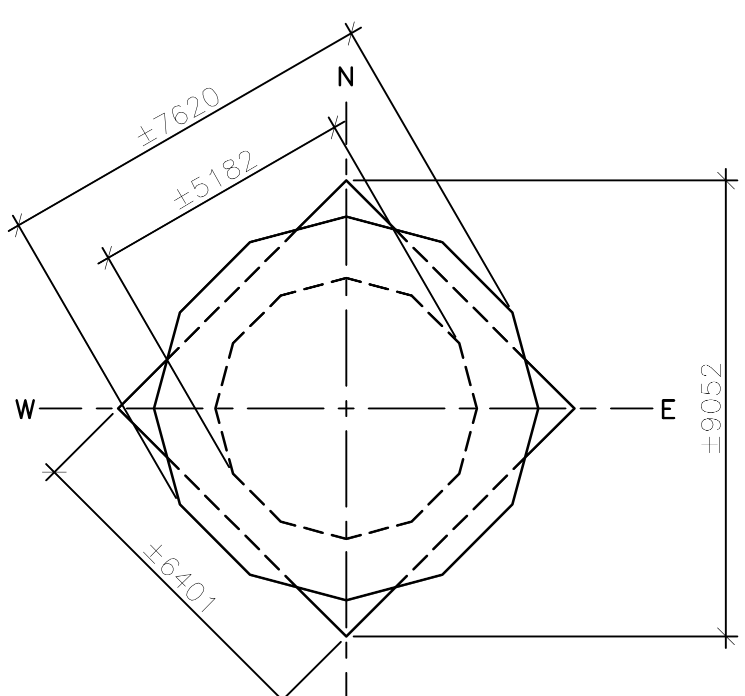


NORTH ELEVATION

EAST ELEVATION

SOUTH ELEVATION

WEST ELEVATION



PIVOT PIER PLAN

No.	Description
A	ISSUED FOR DRAFT REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:150	T.P.	J.N.	T.P.	C.J.	223049.10	

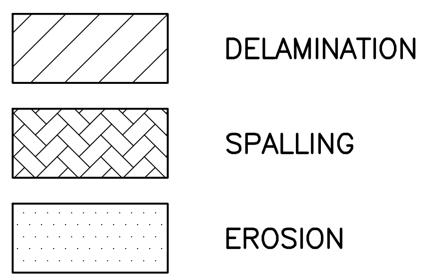
CBCL

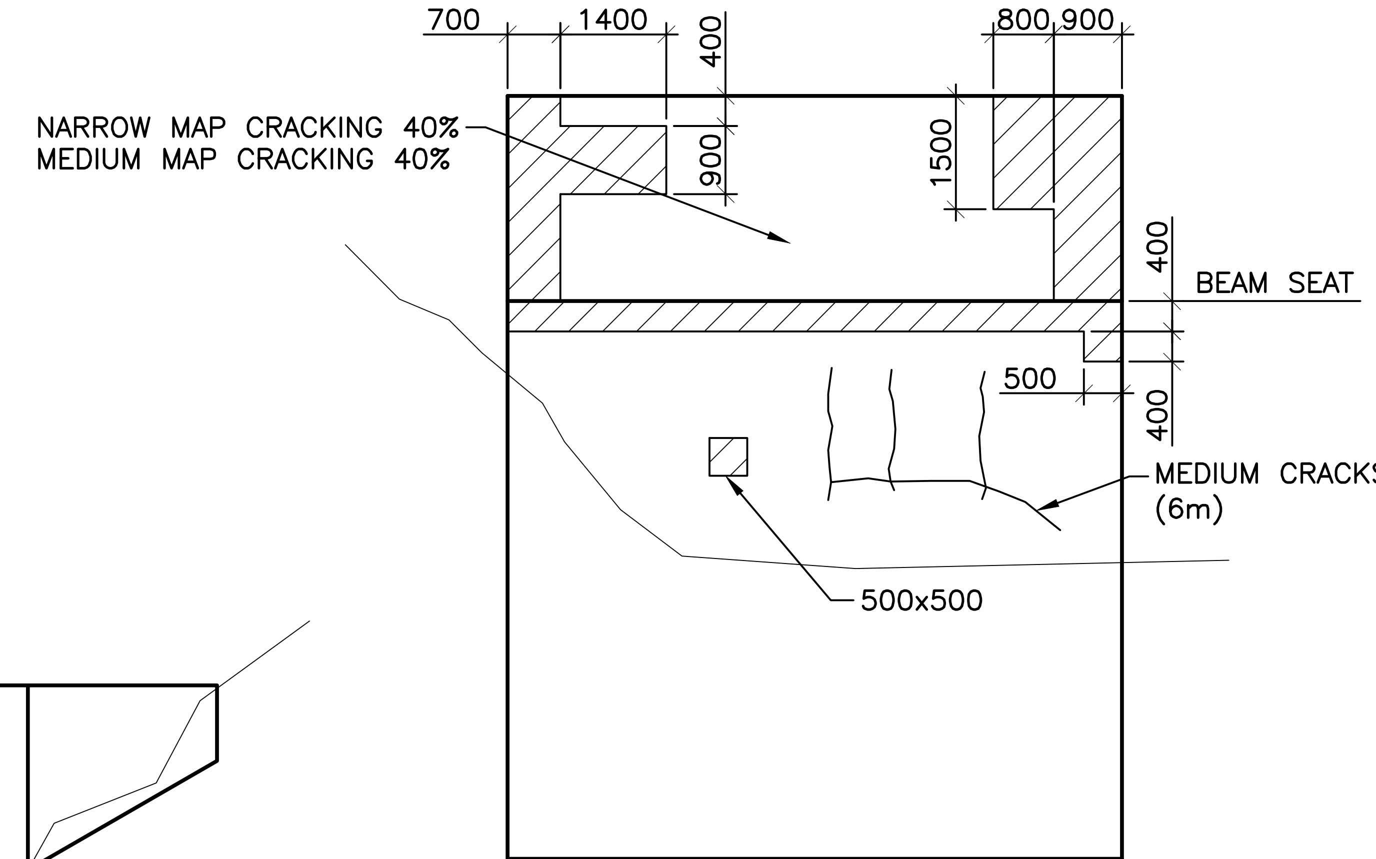
MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

PIVOT PIER - ELEVATIONS

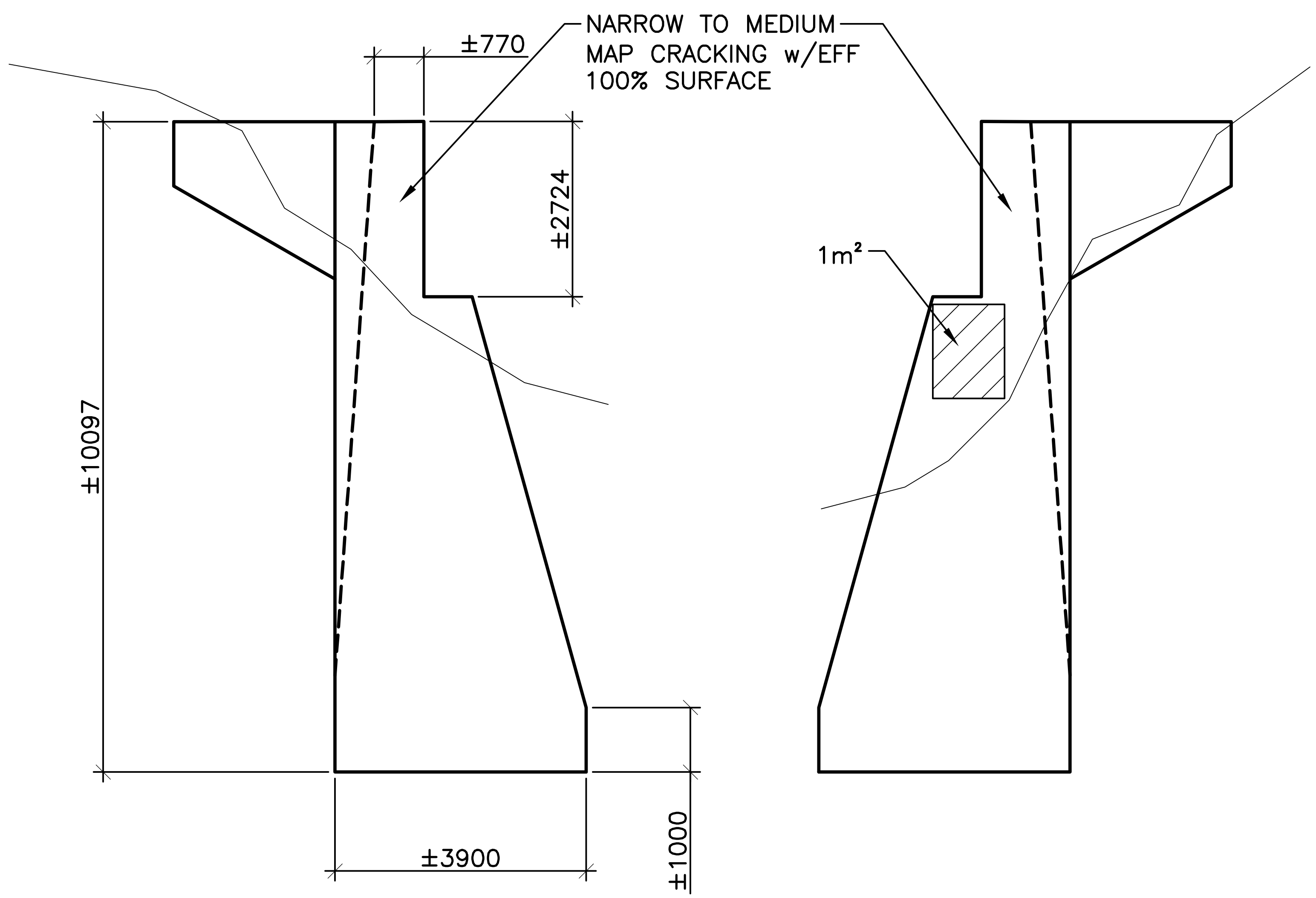
Drawing

S-103



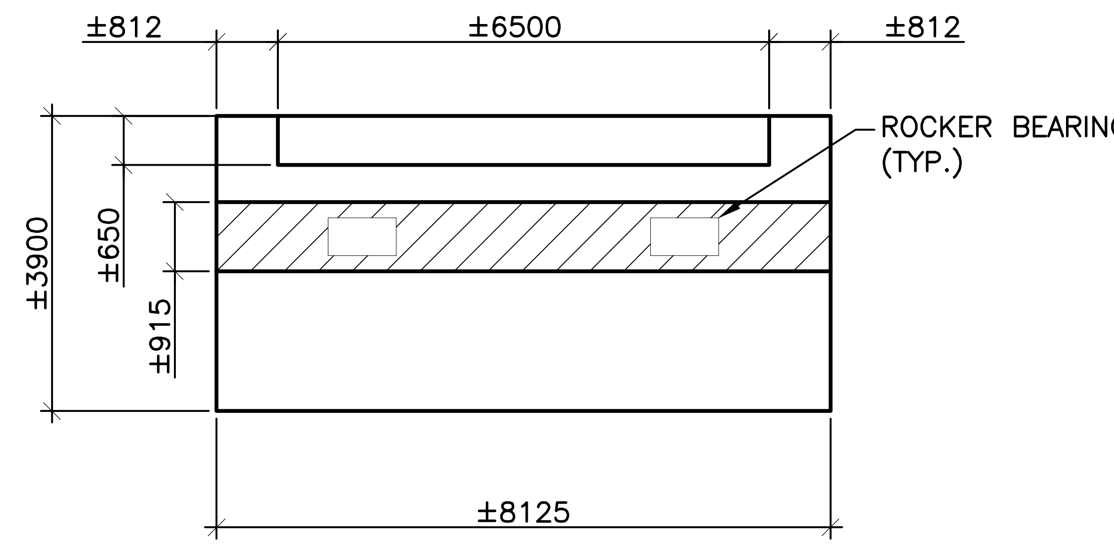


ELEVATION – BACKWALL



ELEVATION – NW WINGWALL

ELEVATION – NE WINGWALL



PLAN – BEAM SEAT

No.	Description
A	ISSUED FOR DRAFT REPORT

Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:100	T.P.	J.N.	T.P.	C.J.	223049.10	

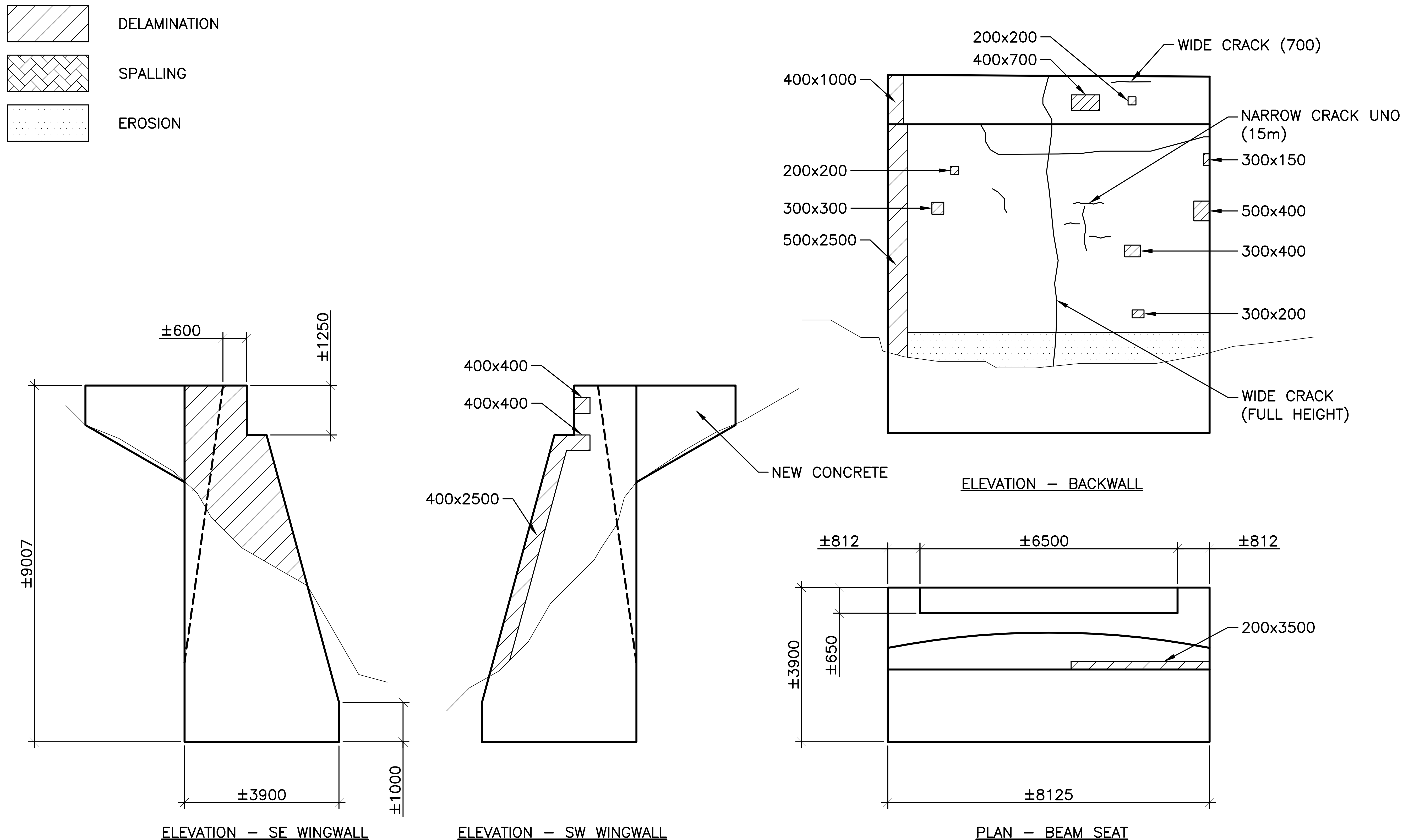
CBCL

MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

NORTH ABUTMENT - ELEVATIONS

Drawing

S-201



Date	Scale	Designed	Drawn	Checked	Approved	CBCL No.	Contract
OCT. 2022	1:100	T.P.	J.N.	T.P.	C.J.	223049.10	

MARYSTOWN HARBOUR BRIDGE - FIELD NOTES

SOUTH ABUTMENT - ELEVATIONS

Drawing
S-202

No.	Description
A	ISSUED FOR DRAFT REPORT

APPENDIX E

Rope Access Visual Assessment Report

ACUREN

Acuren Group Inc.
276 Rothesay Avenue
Saint John, NB, Canada E2J 2B8
www.acuren.com

Phone: 506.633.1774
Toll Free: 800.252.1774
Fax: 506.633.7460

A Higher Level of Reliability

NONDESTRUCTIVE EXAMINATION

CLIENT: CBCL Ltd.

CBCL

DATE: Oct 4th, 2022
ACUREN JOB #: 164-J031765
REPORT #: VT-AG-092922-61
CONTRACT/PO: N/A WO: N/A

ATTENTION: **Mitch Warren**
WORK LOCATION: Marystown NL
PROJECT: Marystown Harbour Bridge Inspection
ITEM(S) EXAMINED: Bridge Structure

PART #: Canning Bridge MATERIAL: Carbon steel THICKNESS: Varying
SCOPE: Perform a visual inspection of Canning bridge
TYPE OF INSPECTION: Visual

TEST DETAILS:

ACCEPTANCE STANDARD: Client's Information REVISION: N/A
PROCEDURE/TECHNIQUE: CAN-VT-17P001 REVISION: 08
METHOD: Direct
EQUIPMENT TYPE: Camera MANUFACTURER: N/A MODEL: N/A S/N: N/A
LIGHT SOURCE: Natural light ILLUMINATION INTENSITY: > 100 fc
LIGHT METER S/N: 2038863 CAL. DUE: JAN 12/23
ADDITIONAL EQUIPMENT: N/A MAGNIFICATION POWER:N/A
SUPPLEMENTAL NDT REPORT ATTACHED?: Yes PROCEDURE DEMONSTRATION REQUIRED?: No
TEST SURFACE CONDITION: 17°C

RESULTS:

As requested, a visual inspection was carried out on Canning bridge via rope access. Results and findings are displayed in photos on the following pages. See below.

Summary of visual inspection

- Splice plates and their components are heavily corroded
- Majority of bearings are heavily corroded
- Holes in lateral beams at swing pier
- Majority of Brackets holding up 4” line on east side of the bridge is detached

CLIENT: __ CLIENT PRINTED NAME CLIENT SIGNATURE ACCEPTED & ACKNOWLEDGED BY DTR NO.: N/A

ACUREN TECHNICIAN: Andrew Goodyear
1st Technician
CGSB UT-1, MT-2
CGSB Reg. #22706
2nd Technician

REVIEWER: Jane Matthews
11/08/22

(Generated Using: CAN-QUA-02F007 R10 - 12/09/2021)

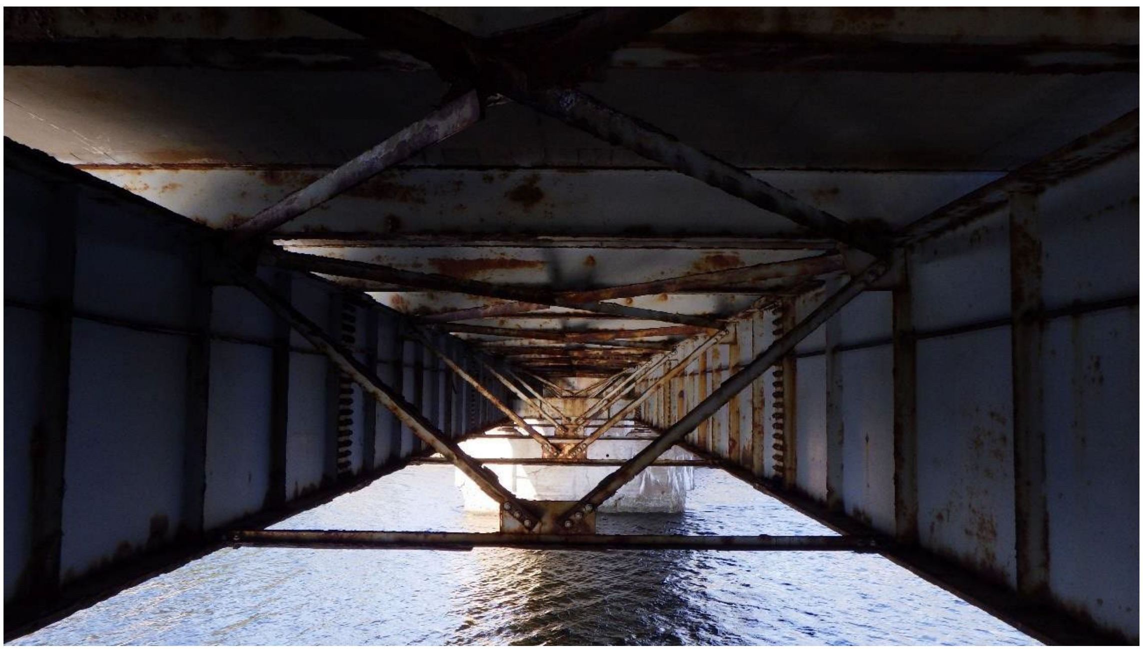


Figure 1. From swing pier looking North

Figure 3. Bearing at swing pier

Figure 4. Bearing at swing pier

Figure 5. Swing stage pier bearing. Severe corrosion

Figure 6. Swing stage pier bearing. Severe corrosion

Figure 7. North side of pier three

Figure 8. East girder

Figure 9. West Girder

Figure 10. From Pier 3 looking at swing pier

Figure 11. Substantial material loss on bearing components. Pier 1

Figure 12. Holes in lateral beam on swing pier.

Figure 13. Majority of brackets holding up 4" line is completely detached from girder.

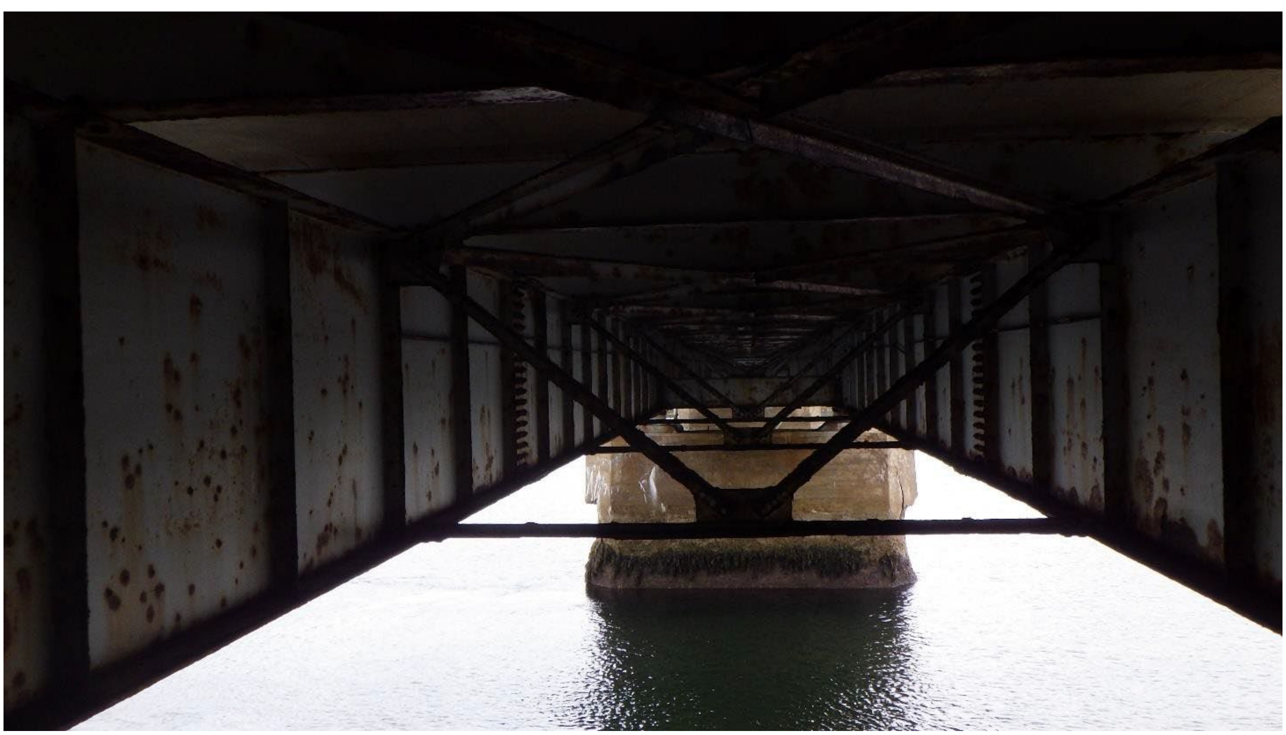


Figure 14. Major corrosion on these braces throughout continuous section of bridge.

Figure 15. Typical condition of splice plate bolts and nuts, throughout whole bridge.

Figure 16. Bracket that has separated from girder.

APPENDIX F

Ultrasonic Thickness Measurement Report

Acuren Group Inc.
2 - 240 Taiganova Crescent
Fort McMurray, AB, Canada T9K 0T4
www.acuren.com
Phone: 780.790.1776
Fax: 780.790.9061

A Higher Level of Reliability

NONDESTRUCTIVE EXAMINATION

CLIENT: CBCL Ltd.

DATE: September 29th, 2022
ACUREN JOB #: 164-J031765
REPORT #: UT-AG-092922-59
CONTRACT/PO: N/A WO: N/A

ATTENTION: Mitch Warren
WORK LOCATION: Marystown NL
PROJECT: Marystown Harbour Bridge Inspection
ITEM(S) EXAMINED: Girders (East and West), Floor Beams, Stringers

TAG #: Canning Bridge
MATERIAL: CS
THICKNESS: Varying
SCOPE: Perform UT thickness readings at requested locations throughout structure.
TYPE OF INSPECTION: Ultrasonic

TEST DETAILS:

ACCEPTANCE STANDARD: Client Info REVISION: N/A
PROCEDURE/TECHNIQUE: CAN-UT-14T001 REVISION: 10

TYPE: Thickness METHOD: Contact

INSTRUMENT: Waygate Technologies MODEL: DMS Go S/N: 223904903 CAL DUE: Mar 28, 2023

CAL. BLOCK: Step Block S/N: 16-1448 CABLE-TYPE: Coaxial LENGTH: 5'
CAL. BLOCK: S/N: COUPLANT: Sonotech - Sono 600

Probe & Technique Details:

	Test Angle (°)	Probe Type	Crystal Size	Freq. (MHz)	Serial Number	Damping Ω	Test From	Reference Reflector	Transfer Value	Reference		Scan dB	Range
										dB	% FSH		
1	0	D798	.200"	5	1146996	N/A	OD	backwall	N/A	59	80	61	2"

TEST SURFACE CONDITION: Coating / Clean Bare Metal TEST SURFACE TEMPERATURE: 17°C

RESULTS:

-All readings recorded in millimeters

-Visibly low areas recorded

Limitations – Some areas chosen for UT are too damaged to obtain readings (See photo on page 4)

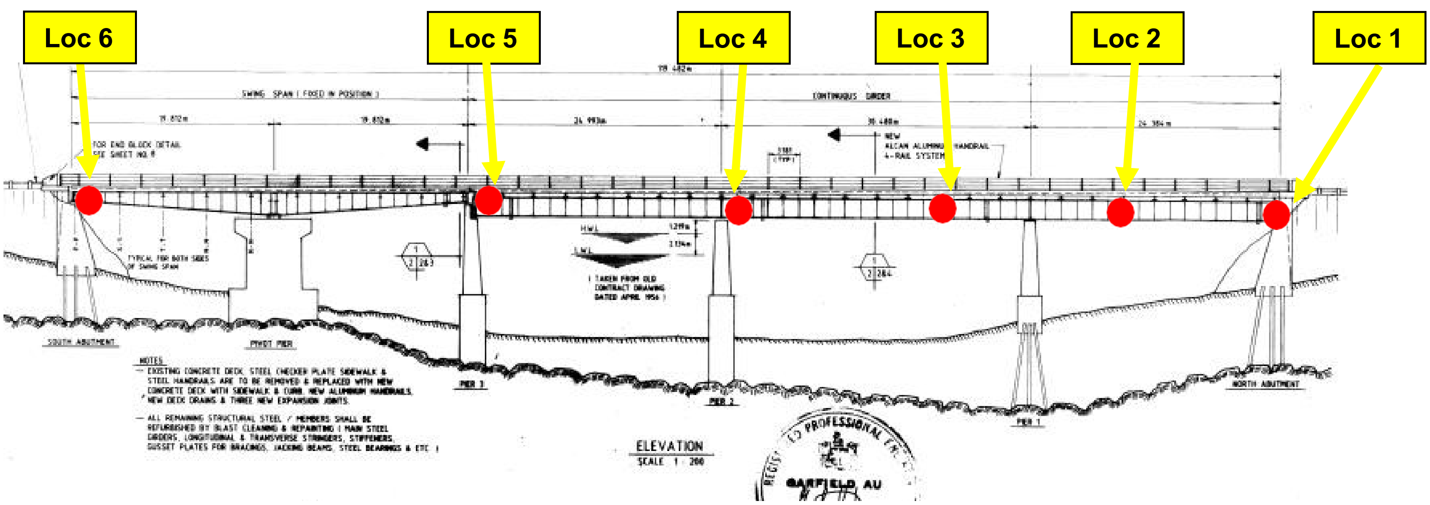
CLIENT: ___
CLIENT PRINTED NAME
CLIENT SIGNATURE
ACCEPTED & ACKNOWLEDGED BY
DTR NO.: N/A

ACUREN TECHNICIAN: Andrew Goodyear
1st Technician
CGSB MT2, UT1 #22706
2nd Technician

REVIEWER: Jane Matthews 11/08/22

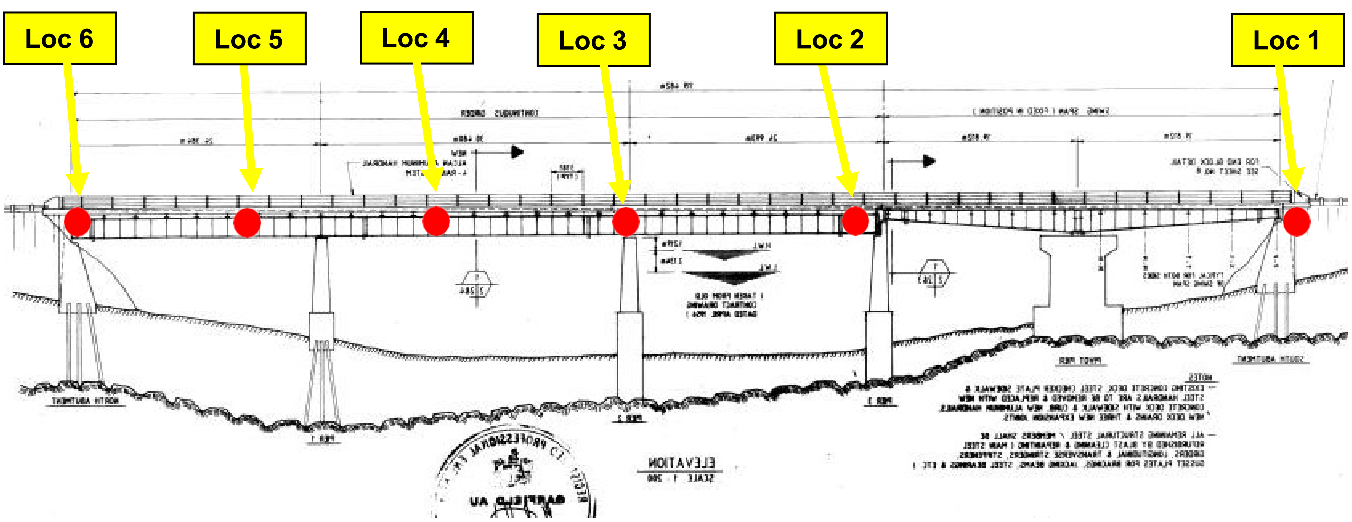
(Generated Using: CAN-QUA-02F007 R10 - 12/09/2021)

Looking West



East Girder			
Location	Top Flange	Webbing	Bottom Flange
UT-1	21.3	10.0	18.9
UT-2	21.6	10.5	19.1
UT-3	20.1	10.7	19.7
UT-4	22.1	10.4	20.5
UT-5	21.0	9.9	21.4
UT-6	22.3	9.6	20.7

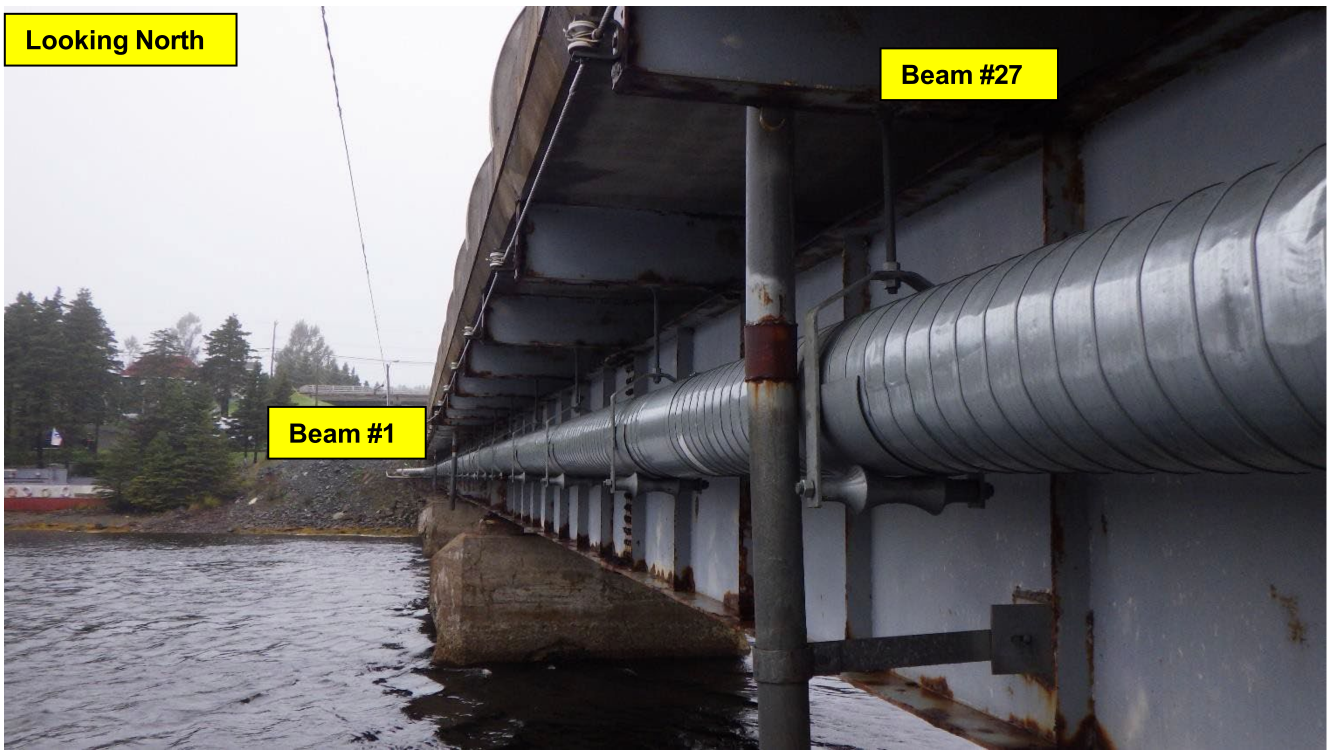
Looking East



West Girder			
Location	Top Flange	Webbing	Bottom Flange
UT-1	20.4	8.4	17.1
UT-2	20.2	11.2	17.7
UT-3	21.0	8.7	19.2
UT-4	21.4	10.8	17.9
UT-5	22.0	9.2	21.1
UT-6	21.7	10.1	20.1

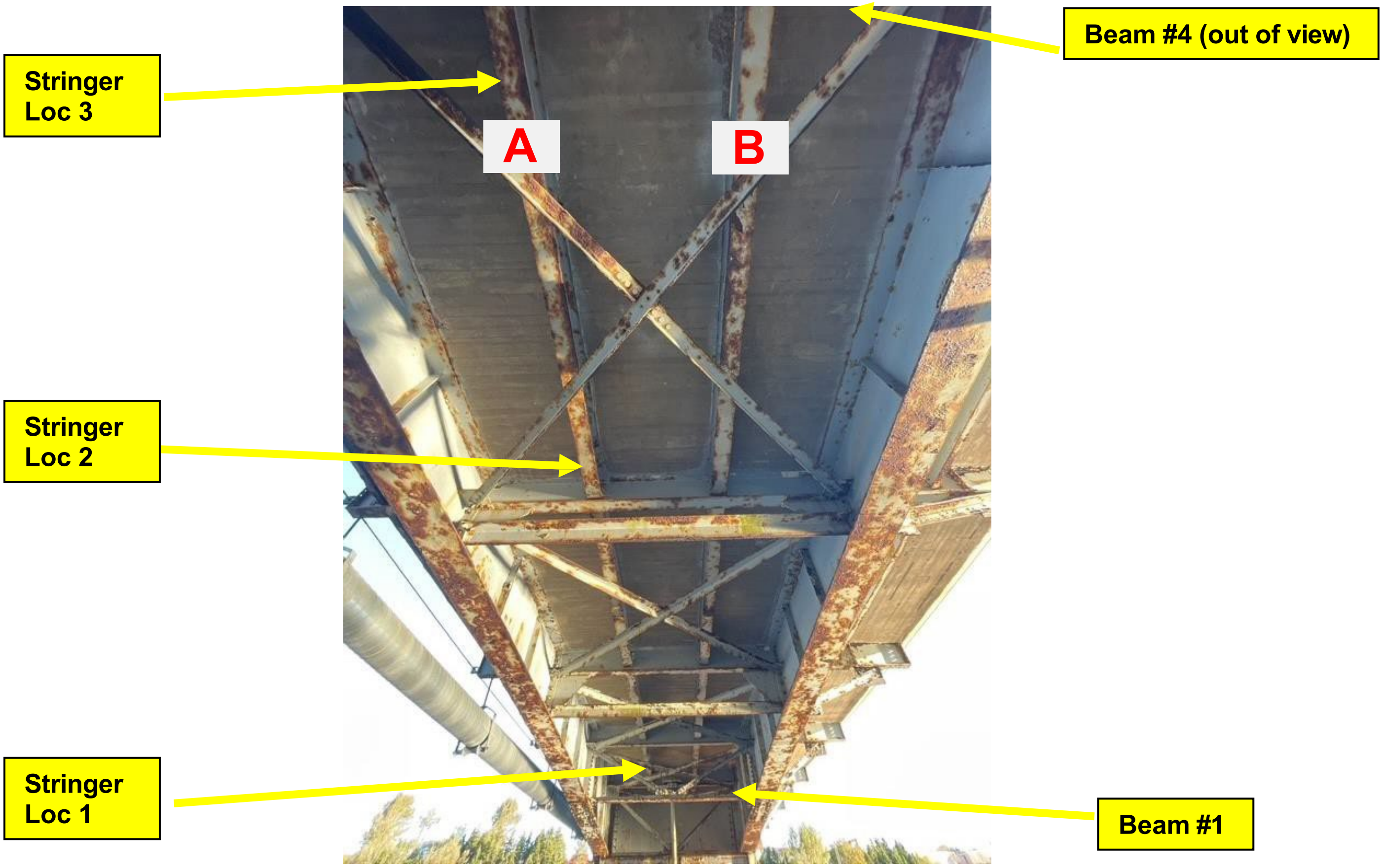
UT Readings not obtainable due to severe material loss/ corrosion

Floor Beams on Continuous Span



Floor Beams on continuous span (27 Total)			
Location	Top Flange	Webbing	Bottom Flange
Beam 3	11.9	9.2	11.2
Beam 9	11.7	8.9	11.5
Beam 16	11.9	9.3	11.1
Beam 22	12.3	9.5	12.3
Beam 27	11.8	9.7	11.6

Stringers and Floor Beams on Swing Span



Floor Beams and Stringers on Swing Span			
Location	Top Flange	Webbing	Bottom Flange
Beam 1	12.2	9.9	11.9
Beam 4	12.5	10.1	11.6
Stringer Loc 1 A	11.9	9.2	11.1
Stringer Loc 1 B	11.7	8.9	11.4
Stringer Loc 2 A	11.7	9.3	11.6
Stringer Loc 2 B	11.4	9.5	12.1
Stringer Loc 3 A	12.1	9.3	11.7
Stringer Loc 3 B	12.3	9.2	11.5

Acuren Group Inc.
1 Austin Street
St. Johns, NL, Canada A1B 4C1
www.acuren.com
Phone: 780.790.1776
Fax: 780.790.9061

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NONDESTRUCTIVE EXAMINATION

CLIENT: CBCL Ltd.

DATE: January 11th, 2023
ACUREN JOB #: 164-J031765
REPORT #: UT-AG-011123-0001 R1
CONTRACT/PO: N/A WO: N/A

ATTENTION: Mitch Warren
WORK LOCATION: Marystown NFLD
PROJECT: Marystown Harbour Bridge Inspection
ITEM(S) EXAMINED: Girder (East), Splice plates (East), See diagrams for specific locations

TAG #: Canning Bridge MATERIAL: CS THICKNESS: Varying
SCOPE: Perform UT thickness readings at requested locations throughout East side of structure.
TYPE OF INSPECTION: Ultrasonic

TEST DETAILS:

ACCEPTANCE STANDARD: Client Info REVISION:
PROCEDURE/TECHNIQUE: CAN-UT-14T001 REVISION: 10
TYPE: Thickness METHOD: Contact
INSTRUMENT: Waygate Technologies MODEL: DMS Go S/N: 223904903 CAL DUE: Mar 28, 2023
CAL. BLOCK: Step Block S/N: 16-1448 CABLE-TYPE: Coaxial LENGTH: 5'
CAL. BLOCK: S/N: COUPLANT: Sonotech - Sono 600

Probe & Technique Details:

	Test Angle (°)	Probe Type	Crystal Size	Freq. (MHz)	Serial Number	Damping Ω	Test From	Reference Reflector	Transfer Value	Reference dB	Reference % FSH	Scan dB	Range
1	0	D798	.200"	5	1146996	N/A	OD	backwall	N/A	59	80	61	2"

TEST SURFACE CONDITION: Coating / Clean Bare Metal TEST SURFACE TEMPERATURE: 0°C

RESULTS:

-All readings recorded in millimetres

Limitations – Floor beams around location 21 are unsafe to climb on, therefore no measurements were taken.

– UT measurements were not taken at section 2 (South), or section (North) due to the time restriction.

Notes – Manual measurements were taken on bottom flange of section 2 (South), but not section 2 (North)

– Manual measurements are extremely inaccurate in some locations due to severe corrosion on top of flange, readings may vary 5-10mm just a couple inches apart. Refer to photos on page 10.

CLIENT: ___
CLIENT PRINTED NAME
CLIENT SIGNATURE
ACCEPTED & ACKNOWLEDGED BY

DTR NO.: N/A

ACUREN TECHNICIAN: Andrew Goodyear
1st Technician
CGSB MT2, UT1 #22706

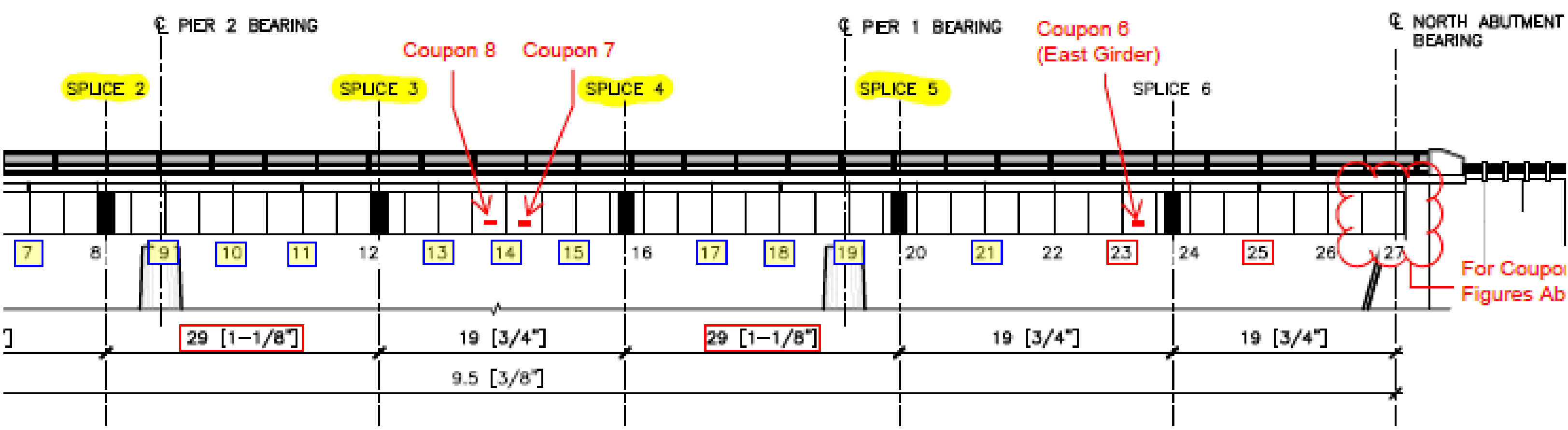
2nd Technician

REVIEWER: Jane Matthews 23/01/23

(Generated Using: CAN-QUA-02F007 R10 - 12/09/2021)

UT Locations on continuous section (Highlighted in blue)

Note- UT Measurements taken precisely where specified.



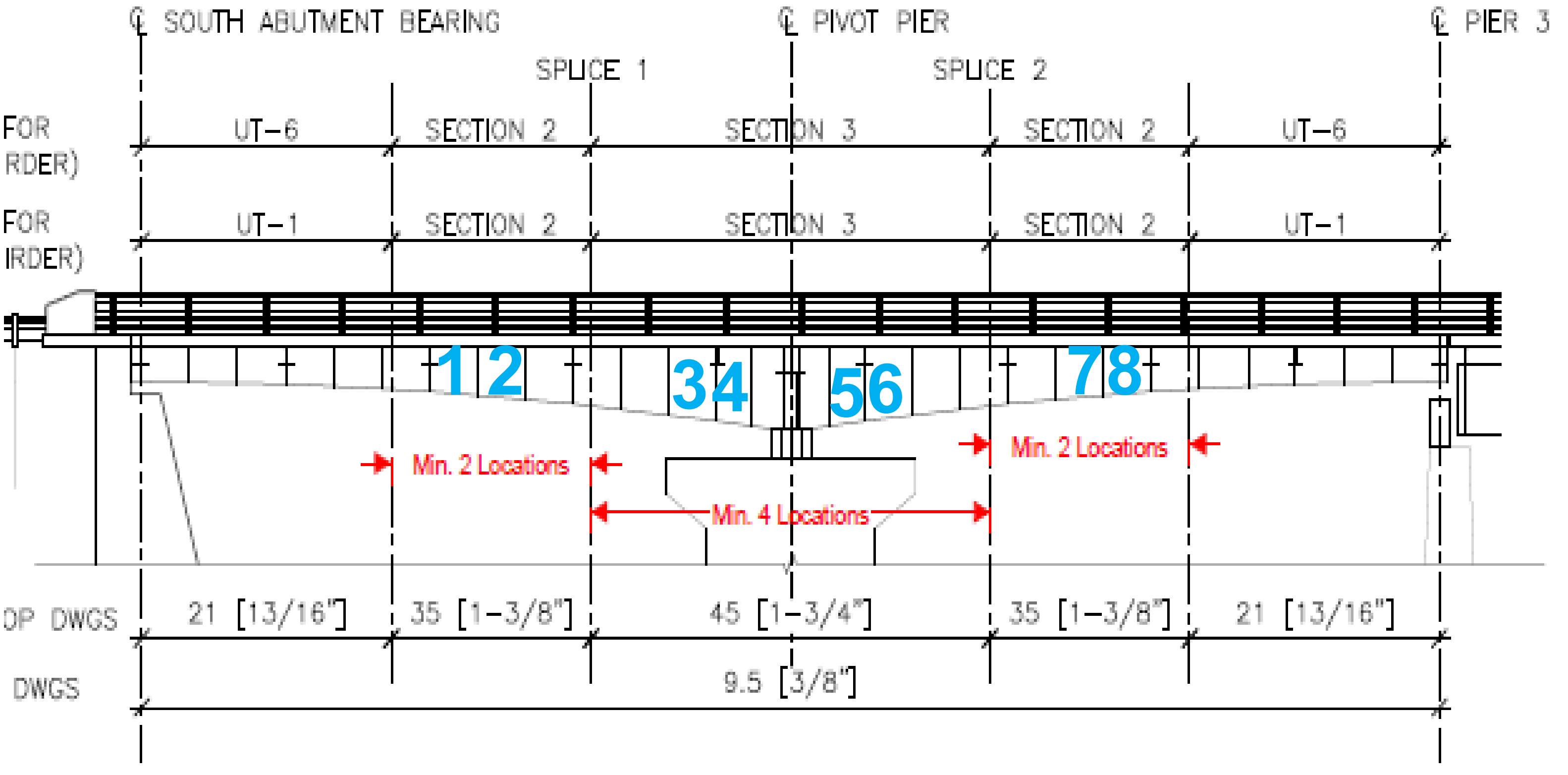
Locations Highlighted in blue (Refer to page 2)

Location 7				
Location	Manual Measurements	UT Measurements		
Top Flange	26.5	19.29	20.21	19.71
Web	N/A	10.58	10.55	10.63
Bottom Flange	22.9	20.45	20.79	20.5
Location 9				
Location	Manual Measurements	UT Measurements		
Top Flange	30	28.06	28.36	27.82
Web	N/A	10.33	10.23	10.22
Bottom Flange	31.2	29.86	29.75	29.79
Location 10				
Location	Manual Measurements	UT Measurements		
Top Flange	30.5	28.39	28.36	28.46
Web	N/A	10.11	10.13	10.1
Bottom Flange	31.8	29.63	29.54	29.5
Location 11				
Location	Manual Measurements	UT Measurements		
Top Flange	33.1	27.51	26.88	26.58
Web	N/A	10.17	10.4	10.39
Bottom Flange	33.4	29.57	29.09	29.19
Location 13				
Location	Manual Measurements	UT Measurements		
Top Flange	22.4	18.54	20	20.14
Web	N/A	11.1	11.36	11.25
Bottom Flange	22.5	21.38	21.67	21.7
Location 14				
Location	Manual Measurements	UT Measurements		
Top Flange	22.1	19.74	19.57	19.16
Web	N/A	11.31	11.12	11.27
Bottom Flange	22.6	20.95	21.16	21.06

Locations Highlighted in blue (Refer to page 2)

Location 15				
Location	Manual Measurements	UT Measurements		
Top Flange	22.3	19.72	20.59	20.58
Web	N/A	11.7	11.47	11.36
Bottom Flange	22.5	22.37	21.58	21.28
Location 17				
Location	Manual Measurements	UT Measurements		
Top Flange	29.04	29.83	28.28	28.31
Web	N/A	10.45	10.4	10.39
Bottom Flange	29.18	30.4	30	29.95
Location 18				
Location	Manual Measurements	UT Measurements		
Top Flange	29.29	28.79	27.8	28.82
Web	N/A	10.27	10.56	10.48
Bottom Flange	29.94	30.44	30.49	30.16
Location 19				
Location	Manual Measurements	UT Measurements		
Top Flange	30.91	32.33	29.21	28.44
Web	N/A	10.56	10.66	10.18
Bottom Flange	30.93	32	31.2	32.25
Location 21				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	No Access		
Web	N/A			
Bottom Flange	N/A			

Locations on swing stage (Locations 1-8)



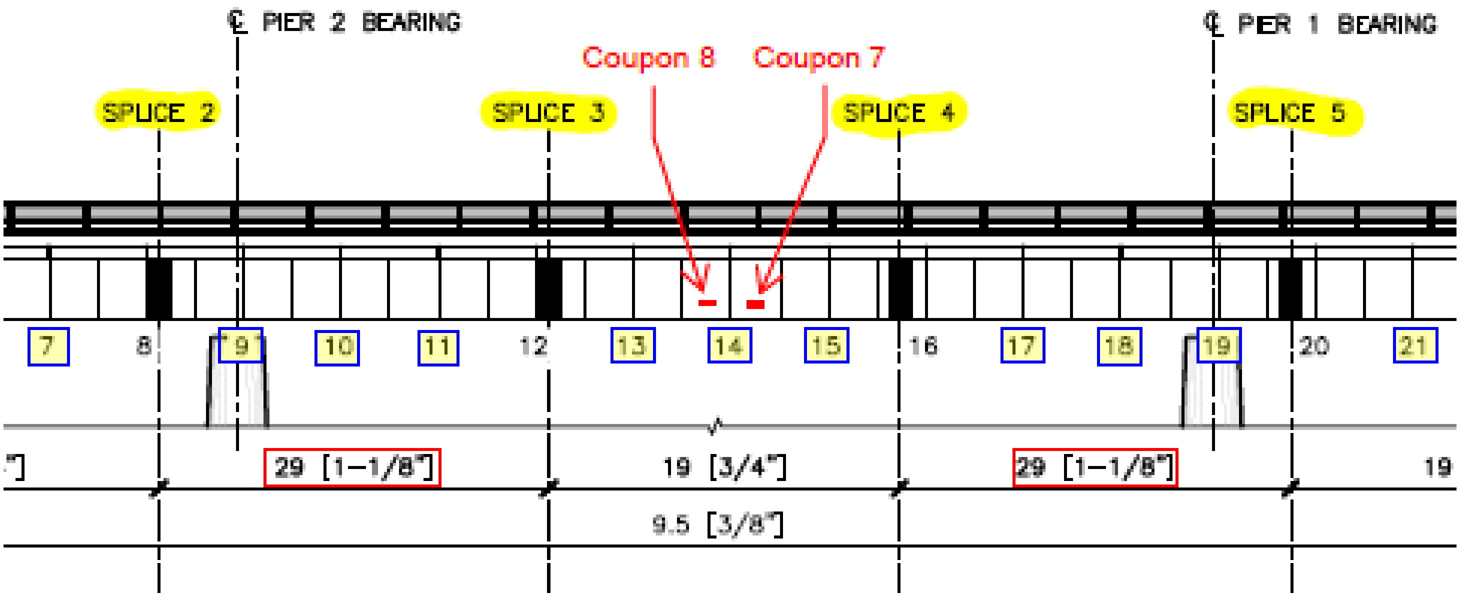
Locations Highlighted in blue (Refer to page 5)

Location 1				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	N/A		
Web	N/A			
Bottom Flange	44.1			
Location 2				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	N/A		
Web	N/A			
Bottom Flange	42.5			
Location 3				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	46.04	46.2	46.19
Web	N/A	10.24	10.23	10.2
Bottom Flange	47.8	45.09	45.08	45.1
Location 4				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	46.24	46.31	46.24
Web	N/A	10.2	10.18	10.2
Bottom Flange	48.4	48.02	47.92	47.82

Locations Highlighted in Blue (Refer to page 5)

Location 5				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	46.87	46.6	46.75
Web	N/A	10.68	10.78	10.86
Bottom Flange	49.4	48.53	48.57	48.44
Location 6				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	46.52	46.53	46.61
Web	N/A	10.26	10.32	10.37
Bottom Flange	48.6	46.61	46.67	46.54
Location 7				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	N/A		
Web	N/A			
Bottom Flange	N/A			
Location 8				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	N/A		
Web	N/A			
Bottom Flange	N/A			

Splice plates (Locations Highlighted in yellow)



Splice plate Locations Highlighted in yellow (Refer to page 8)

Splice 2				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	14.3	14.35	14.4
Web	N/A	12.07	12.11	12.13
Bottom Flange	N/A	15.83	15.9	15.84
Splice 3				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	14.95	15	14.85
Web	N/A	12.35	12.56	12.31
Bottom Flange	N/A	15.47	15.29	15.22
Splice 4				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	14.66	13.83	13.83
Web	N/A	12.58	12.46	12.25
Bottom Flange	N/A	15.43	15.38	15.37
Splice 5				
Location	Manual Measurements	UT Measurements		
Top Flange	N/A	14.22	14.02	13.99
Web	N/A	12.31	12.36	12.34
Bottom Flange	N/A	14.57	14.44	14.24

APPENDIX G

Coating Report

Acuren Group Inc.
1 Austin Street
St. John's, NL, Canada A1B 4C1
www.acuren.com
Phone: 709.753.2100
Fax: 709.753.7011

A Higher Level of Reliability

AMPP COATING EXAMINATION

CLIENT: CBCL Limited
22 King Street PO Box 20040
Saint John
NB
Canada
E2L 5B2

DATE: September 26th, 2022
ACUREN JOB #: 802-J031765
REPORT #: CBCL 0012022
CONTRACT/PO:
WO:

ATTENTION: **MITCHELL WARREN**
WORK LOCATION: Marystown, NL
PROJECT: Campbell Bridge Inspection
ITEM(S) EXAMINED: AMPP Coating Inspection

PART #: n/a
MATERIAL: Steel
THICKNESS: Varies
SCOPE: Visual Inspection and Dry Film Thickness Measurements
TYPE OF INSPECTION: Dry Film Thickness

TEST DETAILS:

ACCEPTANCE STANDARD: Client's Information
REVISION: N/A
PROCEDURE/TECHNIQUE: AMPP Visual Inspection
REVISION:
TEST EQUIPMENT: Dry Film Thickness Gauge
MANUFACTURER / MODEL: PosiTector/DFT 6000
SERIAL NO.: 764705/FS242273
CAL DUE: June 2023

Work Scope:

Tacten was tasked with completing a AMPP (Formally NACE) Inspection on the coating for the Campbell Bridge located in Marystown, Newfoundland and Labrador. The inspection was completed by Andrew Hillyard, AMPP Level 1 via rope access. The took place on September 26th to September 28th, 2022.

CLIENT: CBCL Engineering Limited
CLIENT PRINTED NAME
CLIENT SIGNATURE
ACCEPTED & ACKNOWLEDGED BY

DTR NO.: N/A

ACUREN TECHNICIAN: Andrew Hillyard
1st Technician
, NACE
2nd Technician

REVIEWER: Kyle Kennedy

1. North Abutment to Pier #1

General condition of the coating between the North Abutment and Pier #1. Examples of coating failures are blistering, staining, and undercutting. A test was preformed with a dull scraper and the gray topcoat layer came off and turned white (Fig#5). DFT were taken over six locations, the average DFT 447.04 micrometers.

Fig #1 - Example of Blistering, Staining, undercutting.

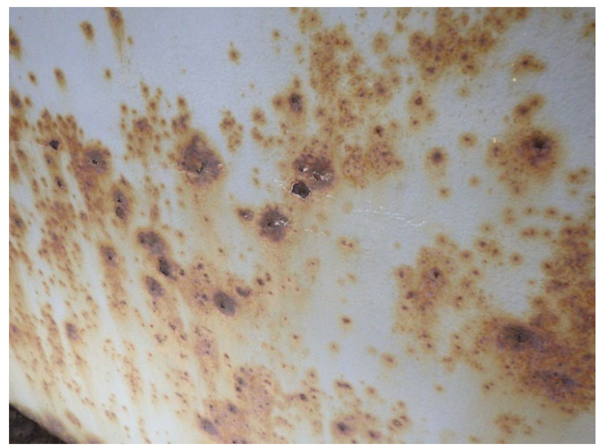


Fig #2 - Example of Blistering, Staining

Fig #3 - Corrosion on the hardware of the splice plate

Fig #4 - Typical delamination of the coating from the substrate

Fig #5 - Scrape test was preformed, showing white layer.

2. <u>Pier #1 to Pier #2</u>

General condition of the coating between Pier #1 and Pier #2. Examples of coating failures are blistering, staining, undercutting, rust spotting, and pinholes. Overall, there is less areas with rust staining then the previous section of the bridge. DFTs were taken over Five locations, the average DFT 740.918 micrometers.

Fig #6 – Typical coating condition on the main bridge beams.

Fig #7 – Typical coating condition on the main bridge beams.

Fig #8 – Typical coating condition on cross bracing.

Fig #9 – Typical coating condition on cross bracing.

Fig #10 – White coating showing through the topcoat.

Fig #11 – Rust spots through all coating layers.

3. Pier #2 to Pier #3

General condition of the coating between Pier #2 and Pier #3. Examples of coating failures are blistering, staining, undercutting, rust spotting, heat damage, pealing. DFT were taken over Five locations, the average DFTs 757.936 mils.

Fig #12 – Rust staining and delamination of coating.

Fig #13 - Example of heat damage, caused by a bracket being welded on the opposite side after initial coating.

Fig #14 - Heavy corrosion shown causing diagonal connection being disconnected from horizontal piece (Fig #15)

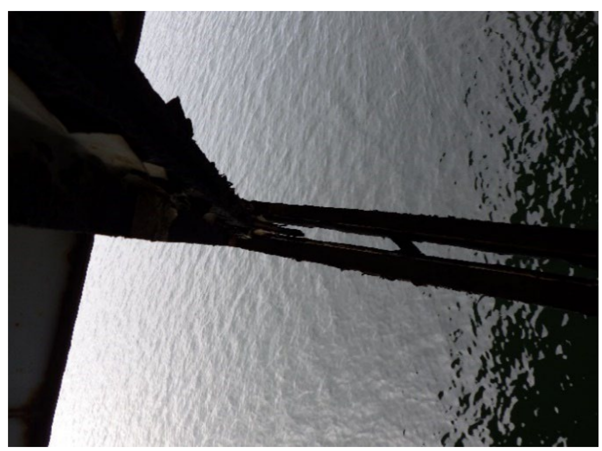


Fig #15

Fig #16 – Swing pier rollers.

Fig #17 - Swing Pier angle steel, delamination of coating.

4. Swing Pier

General condition of the coating of the swing pier structure. Examples of coating failures are blistering, staining, undercutting, rust spotting. A test was preformed with a dull scraper and the gray topcoat layer came off and turned white (Fig#20). DFT were taken over Two locations, the average DFT 632.968 micrometers.

Fig #18 – Typical main beam coating condition.

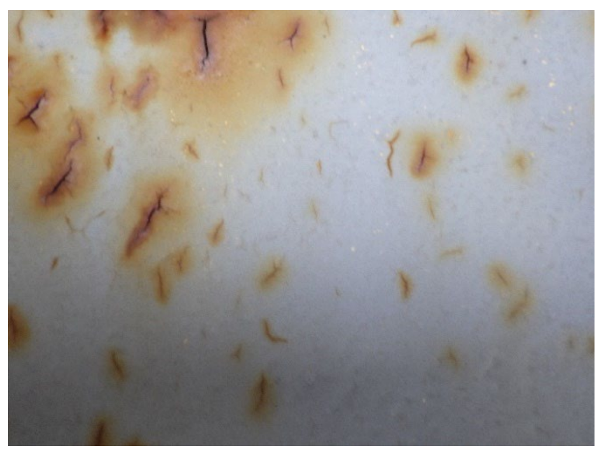


Fig #19 – Rust Blistering and staining.

Fig #20 – Scrap test, dull scrapper.

Fig #21 – Swing Pier pivot structure.

Fig #22 – Inside center support.

Fig #23 – Hole in swing pier structure.

5. **South Abutment**

General condition of the coating on the South Abutment. Examples of coating failures are blistering, staining, undercutting, rust spotting. Average DFT 337.82 micrometers.

Fig #24 – Coating failure, blistering and delamination.

Fig #25 – Rust staining and blistering of coating.

Fig #26 – Main beams, rust staining and blistering.

Summary of Findings

With the current age and environment, the condition is as expected. There is heavy corrosion in areas where water can gather and remain on the structure. Majority of the cross bracings have heavy corrosion, the main support beams show signs of blistering and undercutting on the vertical sections, the top and bottom flanges have high degree of coating failure shown in the above images. The original coating applied to the bridge being unknown it is difficult to know what the original required coating thickness should have been at time of coating; the coating is currently a thickness 330 - 736 micrometers in various areas. The second and Third span show less signs of coating failures in the vertical sections then the first span, this could be attributed to the thickness of the coating in these areas.

North Abutment to Pier 3
Coating Thickness Inspection Report

DeFelsko® Inspection Instruments

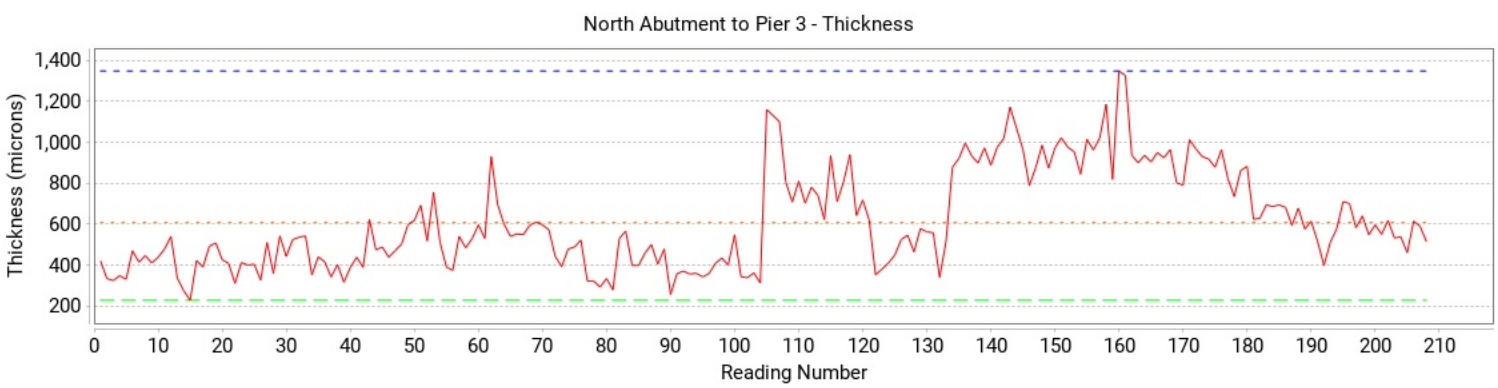
Created:	2022-09-26 07:49:20
PosiTector Body S/N:	764705
Probe Type:	PosiTector 6000 FS
Probe S/N:	242273

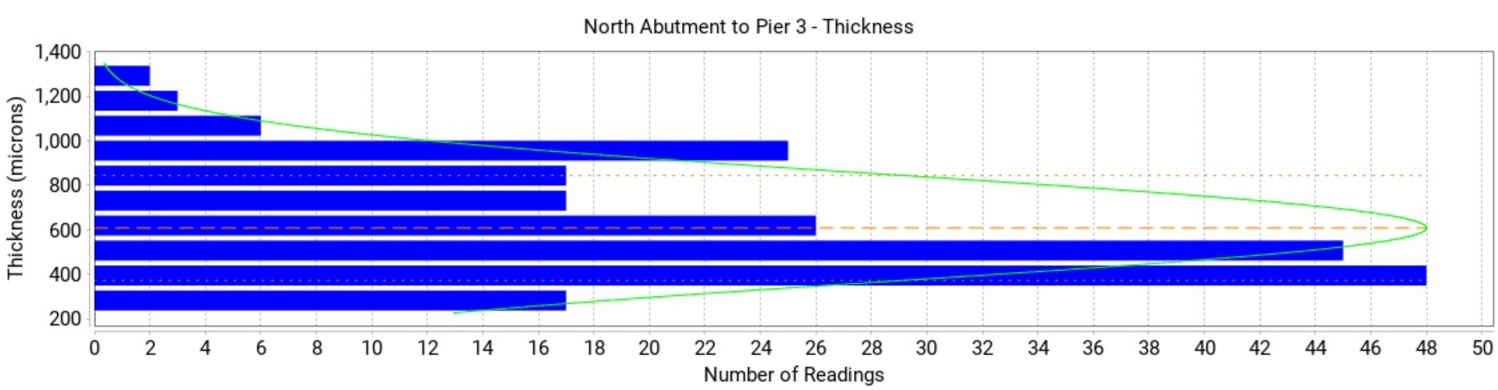
Calibration

Cal Name: Cal 1

Summary

	#	x̄	σ	↓	⤒
	16	611.26	206.99	386.4	967.6
s1	16	386.4	81.3	228	540
s2	10	417.2	54.5	310	508
s3	15	429.1	83.4	316	542
s4	16	517.3	111.4	374	756
s5	16	584.9	108.4	442	930
s6	17	413.2	97.9	256	566
s7	10	393.4	61.5	342	548
s8	9	659.6	336.1	312	1160
s9	11	746.7	111.3	620	940
s10	12	473.0	86.0	340	578
s11	10	965.2	86.8	878	1172
s12	15	950.7	99.7	788	1186
s13	17	967.7	149.8	790	1348
s14	10	730.8	92.8	624	882
s15	13	588.2	85.6	398	710
s16	9	556.9	52.3	460	618





s1 Readings

#	Thickness (microns)
1	420
2	334
3	324
4	348
5	330
6	470
7	414
8	446
9	410
10	438
11	478
12	540
13	336
14	274
15	228
16	392

s2 Readings

#	Thickness (microns)
1	422
2	390
3	492
4	508
5	426
6	408
7	310
8	412
9	400
10	404

s3 Readings

#	Thickness (microns)
1	326
2	510
3	358
4	542
5	442
6	524
7	536
8	542
9	352
10	440
11	416
12	342
13	402
14	316
15	388

s4 Readings

#	Thickness (microns)
1	438
2	388
3	622
4	474
5	488
6	438
7	470
8	502
9	596
10	618
11	692
12	518
13	756
14	512
15	390
16	374

s5 Readings

#	Thickness (microns)
1	540
2	484
3	528
4	596
5	530
6	930
7	696
8	600
9	540
10	552
11	548
12	594
13	610
14	596
15	572
16	442

s6 Readings

#	Thickness (microns)
1	478
2	488
3	522
4	322
5	322
6	292
7	334
8	278
9	530
10	566
11	398
12	398

s6 Readings

#	Thickness (microns)
13	458
14	500
15	404
16	478
17	256

s7 Readings

#	Thickness (microns)
1	358
2	370
3	356
4	360
5	342
6	358
7	408
8	434
9	400
10	548

s8 Readings

#	Thickness (microns)
1	342
2	338
3	362
4	312
5	1160
7	1100
8	804
9	708
10	810

s9 Readings

#	Thickness (microns)
1	702
2	780
3	740
4	622
5	934
6	710
7	806
8	940
9	642
10	718
11	620

s10 Readings

#	Thickness (microns)
1	352
2	380

s10 Readings

#	Thickness (microns)
3	410
4	448
5	524
6	546
7	464
8	578
9	562
10	558
11	340
12	514

s11 Readings

#	Thickness (microns)
1	878
2	920
3	996
4	934
5	898
6	972
7	888
8	976
9	1018
10	1172

s12 Readings

#	Thickness (microns)
1	966
2	788
3	876
4	986
5	874
6	972
7	1022
8	976
9	954
10	844
11	1014
12	962
13	1022
14	1186
15	818

s13 Readings

#	Thickness (microns)
1	1348
2	1326
3	936
4	900
5	936

s13 Readings

#	Thickness (microns)
6	904
7	950
8	924
9	964
10	802
11	790
12	1012
13	968
14	930
15	918
16	878
17	964

s14 Readings

#	Thickness (microns)
1	822
2	734
3	862
4	882
5	624
6	628
7	694
8	686
9	694
10	682

s15 Readings

#	Thickness (microns)
1	594
2	678
3	576
4	614
5	516
6	398
7	512
8	578
9	710
10	700
11	582
12	640
13	548

s16 Readings

#	Thickness (microns)
1	596
2	550
3	618
4	532
5	538

s16 Readings

#	Thickness (microns)
6	460
7	614
8	590
9	514

South Abutment
Coating Thickness Inspection Report

Created:	2022-09-28 07:34:40
PosiTector Body S/N:	764705
Probe Type:	PosiTector 6000 FS
Probe S/N:	242273

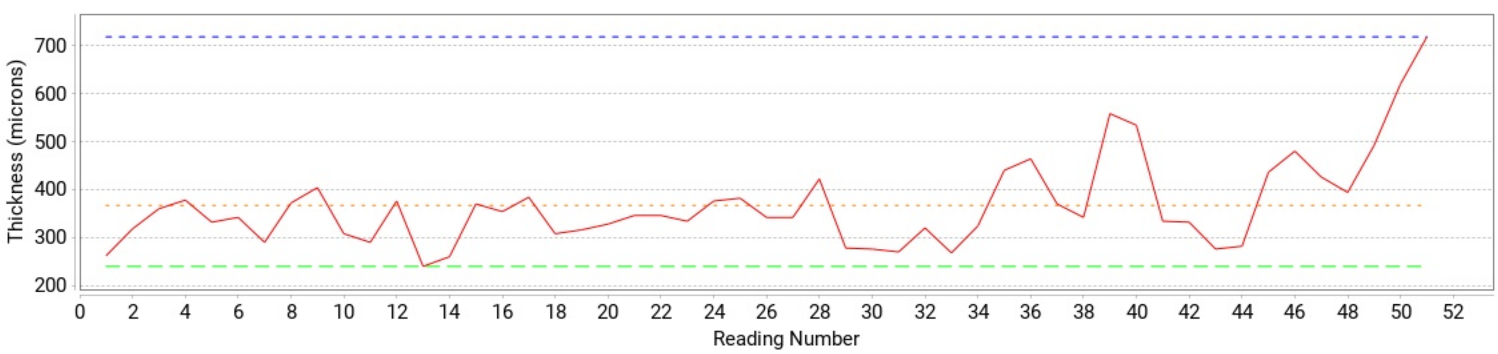
Calibration

Cal Name:	Cal 1

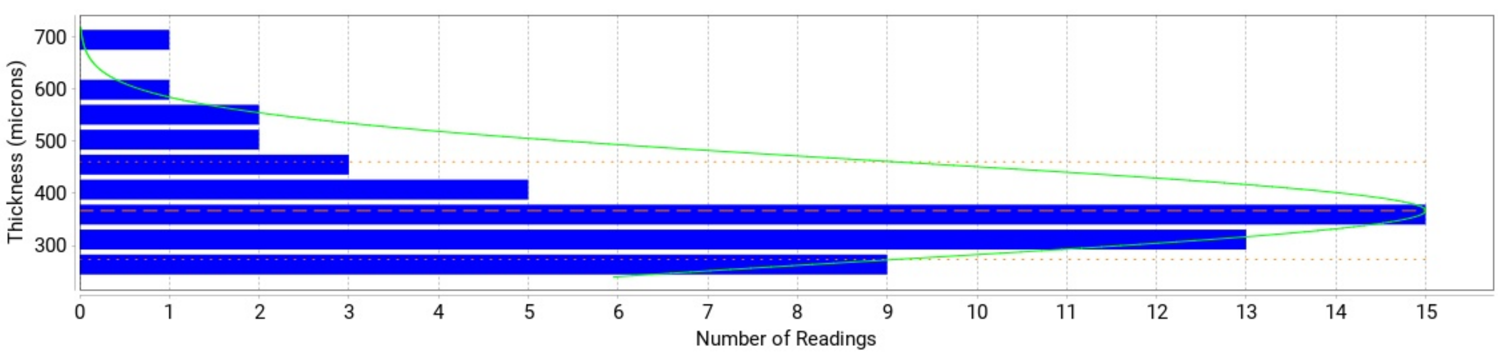
Summary

	#	x̄	σ	↓	⤒
	4	362.42	53.20	326.5	439.6
s1	12	355.8	59.9	262	464
s2	12	326.5	45.6	240	384
s3	12	327.8	49.3	268	422
s4	15	439.6	127.7	276	718

South Abutment - Thickness



South Abutment - Thickness



s1 Readings

#	Thickness (microns)
1	262
2	318
3	360
4	378
5	332
6	342
7	290
8	372
9	404
10	308

DeFelsko®
Inspection Instruments

s1 Readings

#	Thickness (microns)
11	440
12	464

s2 Readings

#	Thickness (microns)
1	290
2	376
3	240
4	260
5	370
6	354
7	384
8	308
9	316
10	328
11	346
12	346

s3 Readings

#	Thickness (microns)
1	334
2	376
3	382
4	342
5	342
6	422
7	278
8	276
9	270
10	320
11	268
12	324

s4 Readings

#	Thickness (microns)
1	370
2	342
3	558
4	534
5	334
6	332
7	276
8	282
9	436
10	480
11	426
12	394
13	492
14	620

s4 Readings

#	Thickness (microns)
15	718

Swing Pier
Coating Thickness Inspection Report

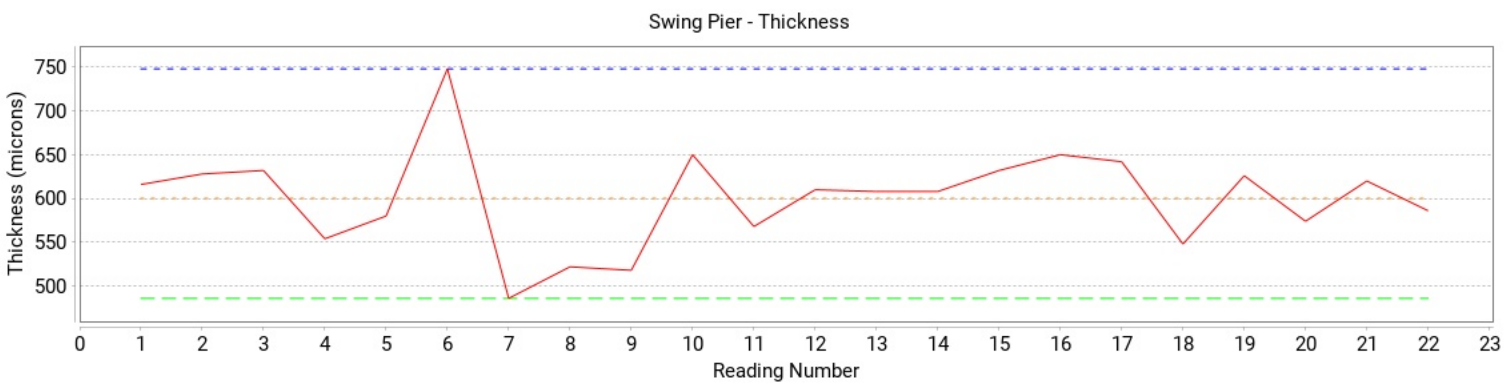
Created:	2022-09-27 07:50:59
PosiTector Body S/N:	764705
Probe Type:	PosiTector 6000 FS
Probe S/N:	242273

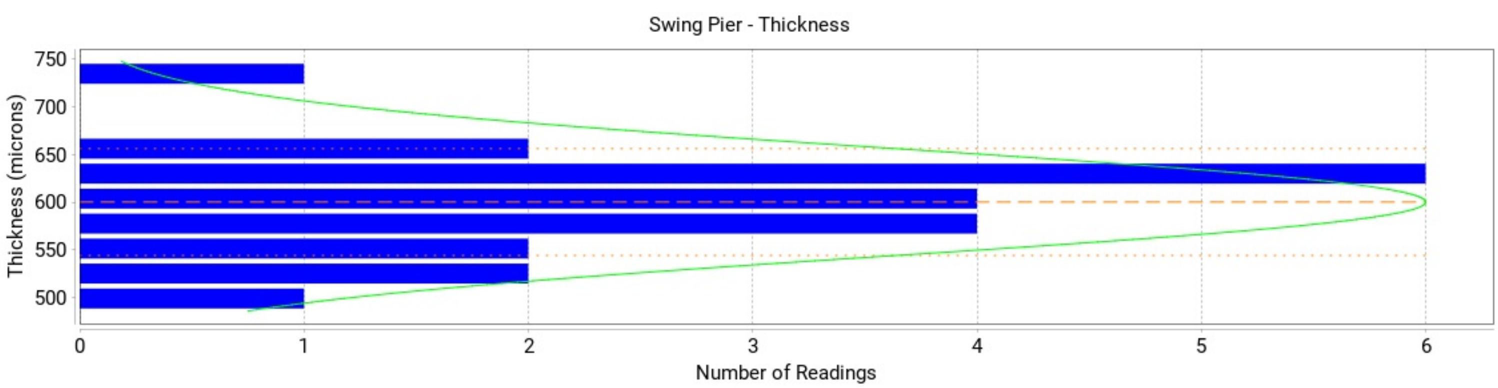
Calibration

Cal Name:	Cal 1

Summary

	#	$\bar{x}$	σ	↓	↑
	2	600.30	13.01	591.1	609.5
s1	11	591.1	74.1	486	748
s2	11	609.5	30.3	548	650





s1 Readings

#	Thickness (microns)
1	616
2	628
3	632
4	554
5	580
6	748
7	486
8	522
9	518
10	650
11	568

Swing Pier
Coating Thickness Inspection Report

DeFelsko® Inspection Instruments

s2 Readings

#	Thickness (microns)
1	610
2	608
3	608
4	632
5	650
6	642
7	548
8	626
9	574
10	620
11	586

APPENDIX H

Dive Inspection Report

CANNING BRIDGE INSPECTION

SEA FORCE DIVING
24 Dundee Avenue, Mount Pearl, NL A1N 4R7

DATE OF SURVEY:

September 26-28, 2022

LOCATION:

Canning Bridge, Marystown, Newfoundland

REPORT WRITTEN BY:

Tony O’Driscoll, Dive Superintendent
Sea-Force Diving Ltd.

DIVING CREW:

Paul Sullivan, Supervisor
Tim Knight, Supervisor/Diver
Justin Bailie, Diver
Chris O’Driscoll, Diver
Andrew Knickle, Dive Tender
Tony O'Driscoll, Superintendent/Vessel Operator

WEATHER CONDITIONS: (Typical day September 27)

Temperature: $+12^0$C
Wind: Light/Variable
Visibility: Overcast
Tide: 1.0m-2.0m (referenced from waterlevels.gc.ca)

UNDERWATER CONDITIONS:

Temperature: $+12^0$C
Visibility 3m
Current: Strong Tidal in a narrow channel

INTRODUCTION:

A diving crew was mobilized to Canning bridge to perform a conditional survey on the three support piers and pivot pier. This survey was conducted over two days to capitalize on slack tide changes to perform dives. The first dive was conducted on Pivot Pier on the south side of the Canning Bridge. The second dives were conducted on Pier one & two with pier three being completed on the third dive. The results of each pier and Pivot pier are presented below and are noted in an electronic PDF file. The diving inspection entailed documenting deterioration, spalling, undermining and general wear to the concrete face. Please reference the attached PDF on piers and pivot. (select the drawing and right click then open to view comments on each pier) The results of the survey are listed in this report.

SURVEY:

1. **Pivot Pier:**

On the Pivot Pier or Swing Pier there are significant signs of undermining and spalling all around the structure as well as concrete debris on the seabed especially concentrated on the east end of the Pier. Compared to Pier 1,2 & 3 the Pivot appears to be in the worst condition with regards to undermining and spalling.

The west end of the Pivot Pier base is undermined on the steel formwork on the footing approximately 300mm long x 130mm wide.
The northwest side of has undermining between 300-450mm long to about halfway down the North side

The south side has undermining approximately 600mm from the bottom of the footing to the seabed. by approximately 1mtr long. This face has significant spalling and more undermining as you approach the east end of the pier.

On the East end, the diver was instructed to try and chip off some concrete and it was found to be easily done.
Significant undermining was located on the east end measuring Approximately 500mm from the seabed to the bottom of the footing. Two meters from the east end going up the north side there is undermining reaching approximately 220mm from the seabed to the bottom of the footing and 600mm long. Also, on the East, an exposed I-Beam is sitting horizontally approximately 1.5Mtr in length.

On the North West end, there is significant spalling
on the footing of 100mm plus.
The Northside has the most significant amount of undermining.

2. Pier 1:

On the Northeast side of Pier Base 1, there is some wooden formwork still embedded in the concrete footing approximately 1.5 meters long running from the center of the north side toward the east end. All along the North side running down and across the east end there are no major signs of spalling and there are no signs of undermining.

On the southeast corner, there are some signs of spalling and an area of undermining running up the south side measuring 360mm long from the SE corner x 200mm in height x 230mm deep into the pier base footing. Along the middle of the Pier base on the south side, there are signs of spalling measuring 460mm Long x 300mm deep x 290mm in height. (multiple pieces of concrete debris on the seabed in this area).

Also, 360 degrees around the pier base you can see a joint left from the pour measuring approximately 130mm deep x 120mm Wide.

At the time of the dive, the water depth at the east end was 6m and the depth at the west end was 5.2m with both having signs of major spalling in approximately 2-2.4m of water.
September 27, 2022

**heavy marine growth and light signs of calcium deposits on the concrete surface

3. Pier 2:

On Pier Base 2 there is no sign of undermining 360 degrees around the Pier base footing.

The Northside has spalling all along its side with concrete debris on the seabed. Along the North side, there is an I-Beam embedded in the footing running from the east end up the North side measuring approximately 3 meters long.

On the East end approximately 6m from the surface there are signs of major spalling. (8.5m at the time of the dive)
All along the South side there are signs of spalling the full length of the footing. (Concrete debris on the seabed).

On the west end approximately 2m there are major signs of spalling. (9m at the time of dive)
*On the northwest corner there is an I-Beam projecting straight out of the footing and measuring approximately 1.5 meters.
* heavy marine growth and light signs of calcium deposits on the concrete surface
September 27, 2022

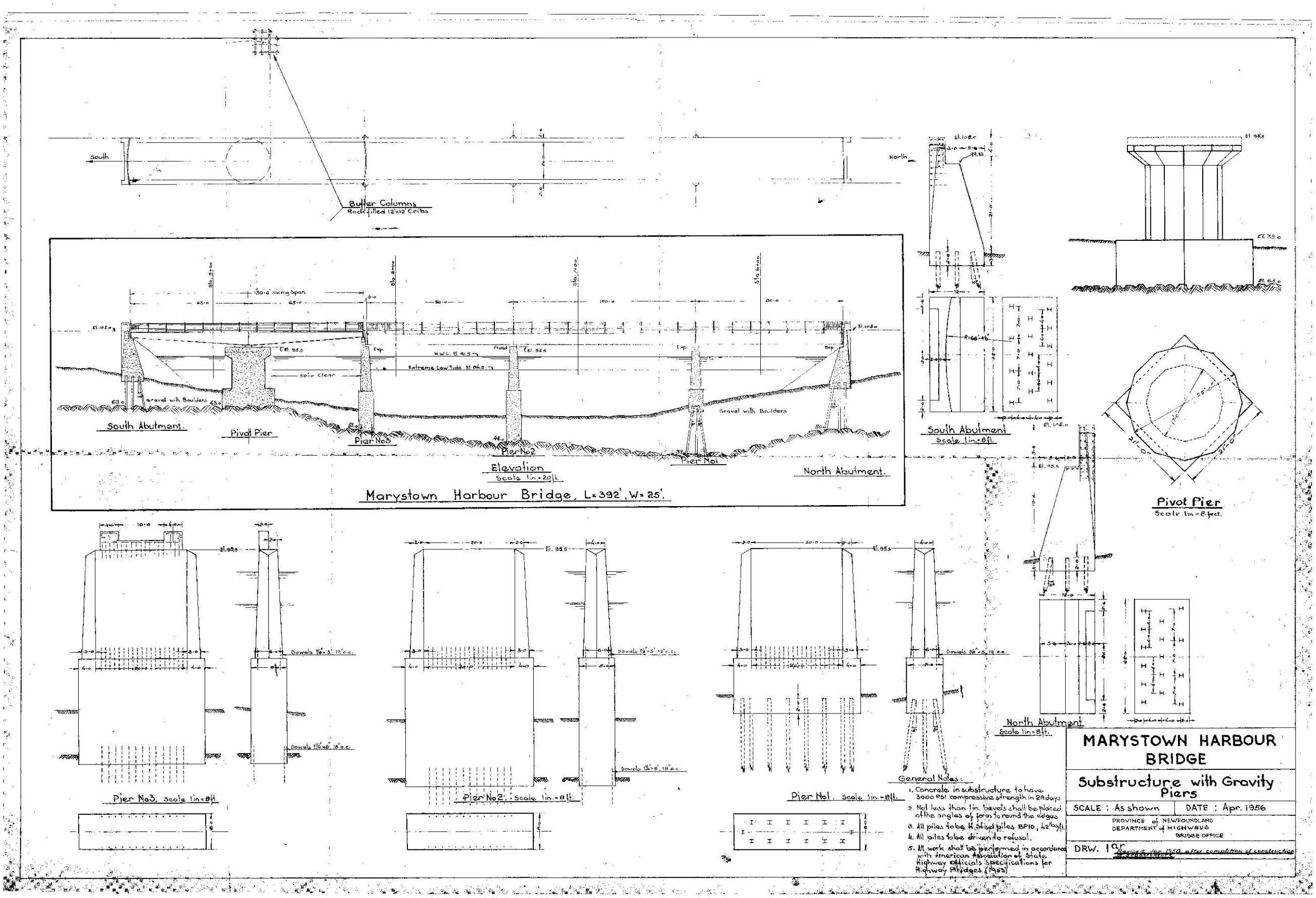
4. Pier 3:

On Pier Base 3 there are no signs of undermining 360 degrees around the Pier base footing and heavy marine growth all over.

On the south side, there are major signs of spalling with concrete debris on the seabed.

There is a crack running up the middle of both sides of the pier base:

- On the South side, it starts approximately 300mm from the footing and travels to the surface measuring approximately 2mm width from the start and upwards of 50mm width as you reach the surface. There is a piece of wood embedded in the south side footing and just appears to be some formwork left behind during the construction phase
- On the North, the crack starts approximately 1 meter from the footing and travels to the surface measuring approximately 5mm width at the start and upwards of 20mm as you reach the surface. The water depth on the East end was 7.3m and on the West end it was 6.7mm at the time of the dive.



PHOTOS:

A series of photographs of the structure were taken and are included in this report as a separate file. Several additional pictures have been provided for reference of typical site conditions.

VIDEO:

A series of videos of each face were conducted and are also included with this report as a separate file.

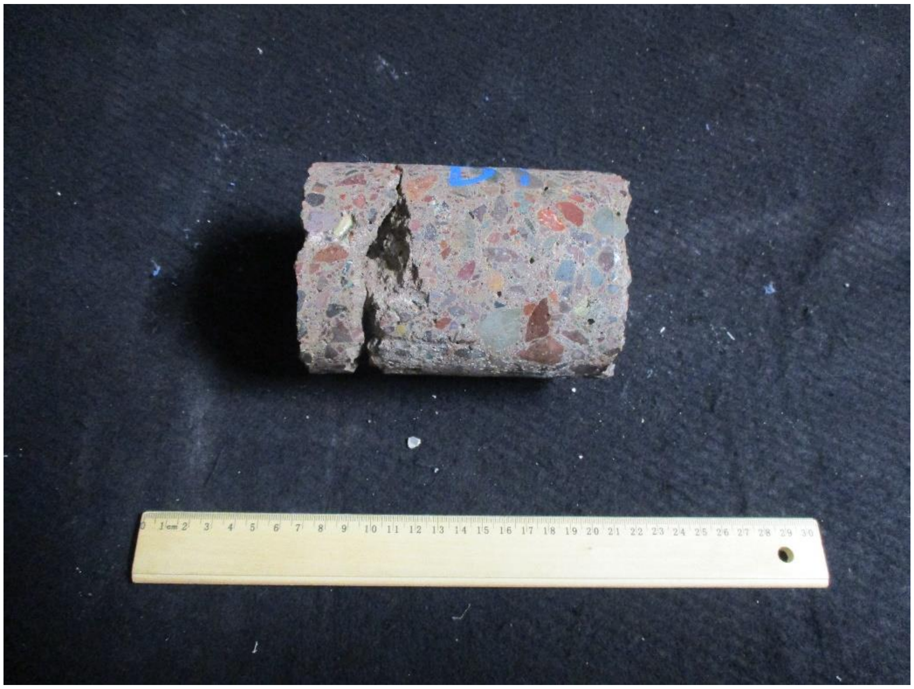
CONCLUSION:

If you have any further concerns regarding this report and/or survey please contact Tony O'Driscoll by phone at (709) 753-2021, cell phone at (709) 687-8123, and by facsimile (709) 753-2035 or by e-mail tony@seaforcediving.com & supervisor@seaforcediving.com.

APPENDIX I

Concrete Material Test Results

Core Photo Log



Core D1
Sampled from Bridge Deck

Core D2
Sampled from Bridge Deck

Core D3
Sampled from Bridge Deck

Core D4
Sampled from Bridge Deck

Core D5
Sampled from Bridge Deck

Core D7
Sampled from Bridge Deck

Core D8
Sampled from Bridge Deck

Core D9
Sampled from Bridge Deck

Core 9
Sampled from South Abutment – Back Wall

Core 11
Sampled from South Abutment – East Wing Wall

Core 12
Sampled from South Abutment – Beam Seat

Core 13
Sampled from South Abutment – Foundation. Approximately Middle

Core 14
Sampled from South Abutment – Foundation. Near Ground Level

Core 15
Sampled from North Abutment – Back Wall

Core 16
Sampled from North Abutment – Back Wall. Approximately Middle

Core 17
Sampled from North Abutment – West Wing Wall

Core 18
Sampled from North Abutment – Top Face Bearing Seat. East End

Core 19
Sampled from North Abutment – Top Face Bearing Seat. Middle

Core 20
Sampled from North Abutment – West half of foundation

Core 21
Sampled from Pier 2 – Above high-waster line

Core 22
Sampled from Pier 2 – Top face

Core 23
Sampled from Pier 3 – Top face

Core 24
Sampled from Pier 3 – Above high-water line

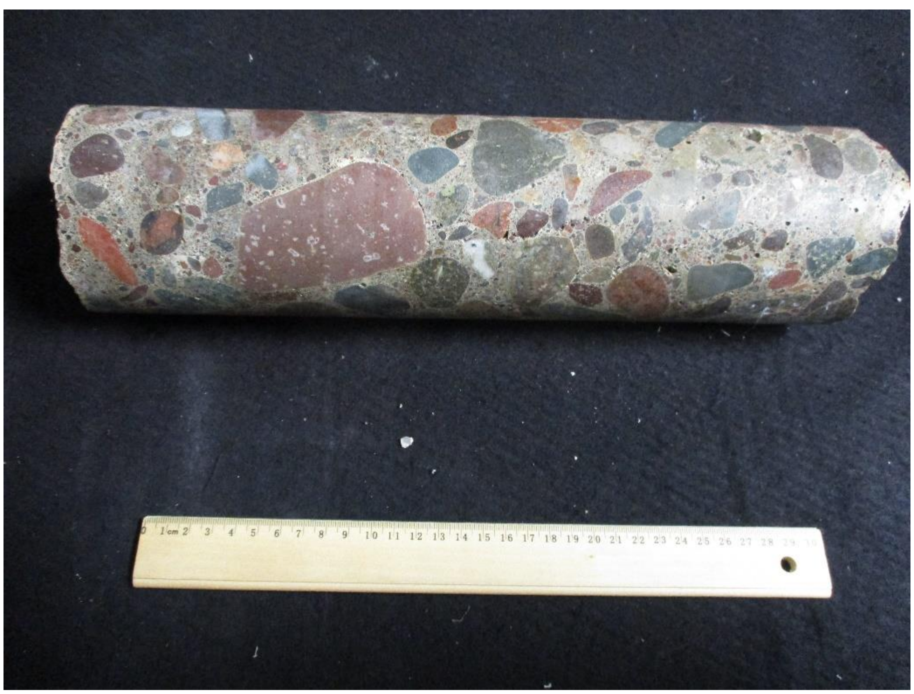
Core 25
Sampled from Pier 3 – Below high-water line in tidal zone

Core 26
Sampled from Pier 1 – Below high-water line in tidal zone

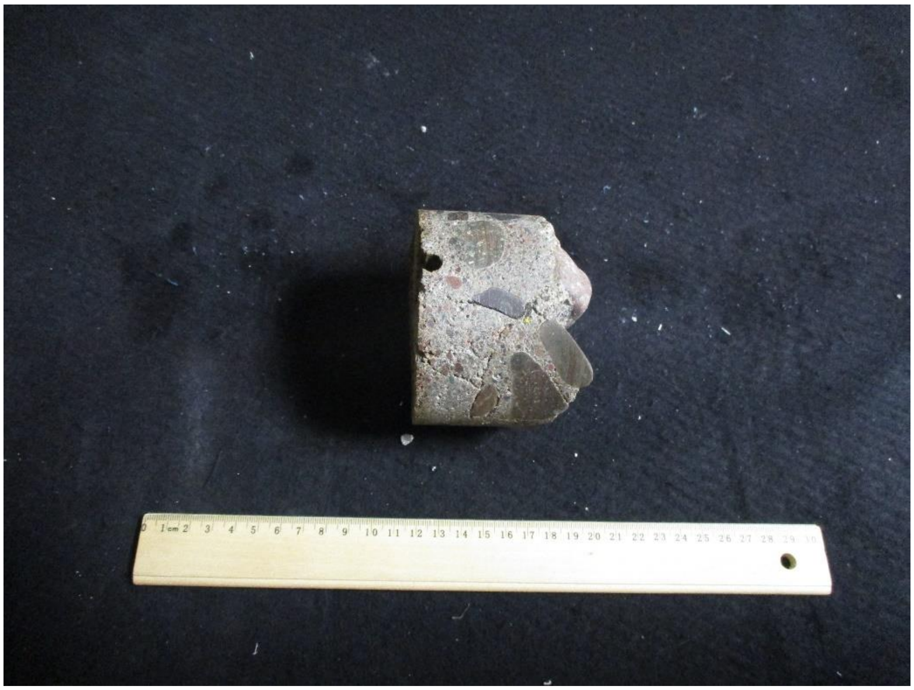
Core 27
Sampled from Pier 1 – Above high-water line

Core 28
Sampled from Pier 1 – Above high-water line

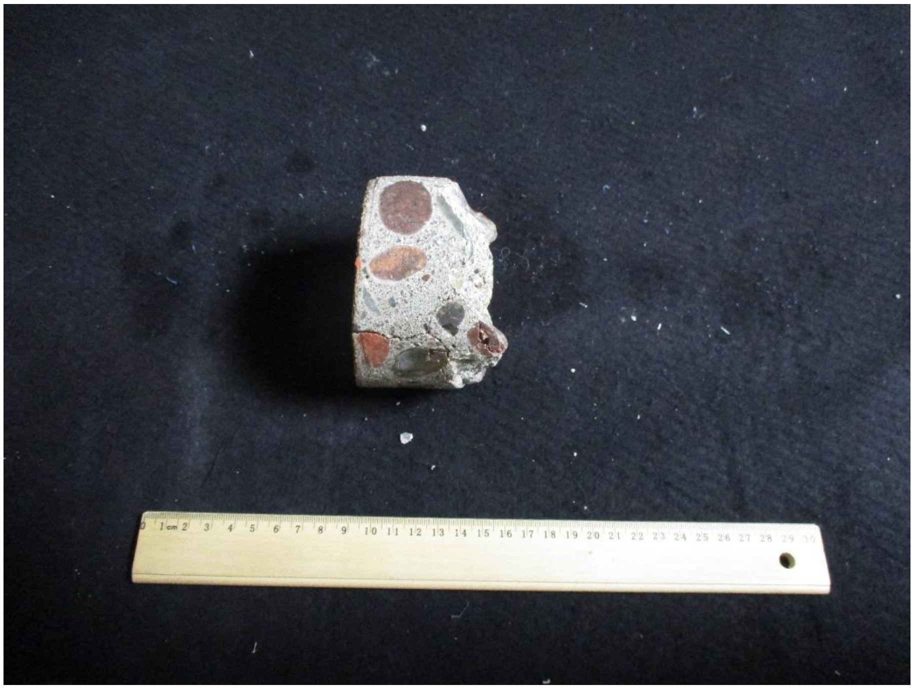
Core 29
Sampled from Pier 1 – Below high-water line in tidal zone



Core 30
Sampled from Pier 1 – Top face



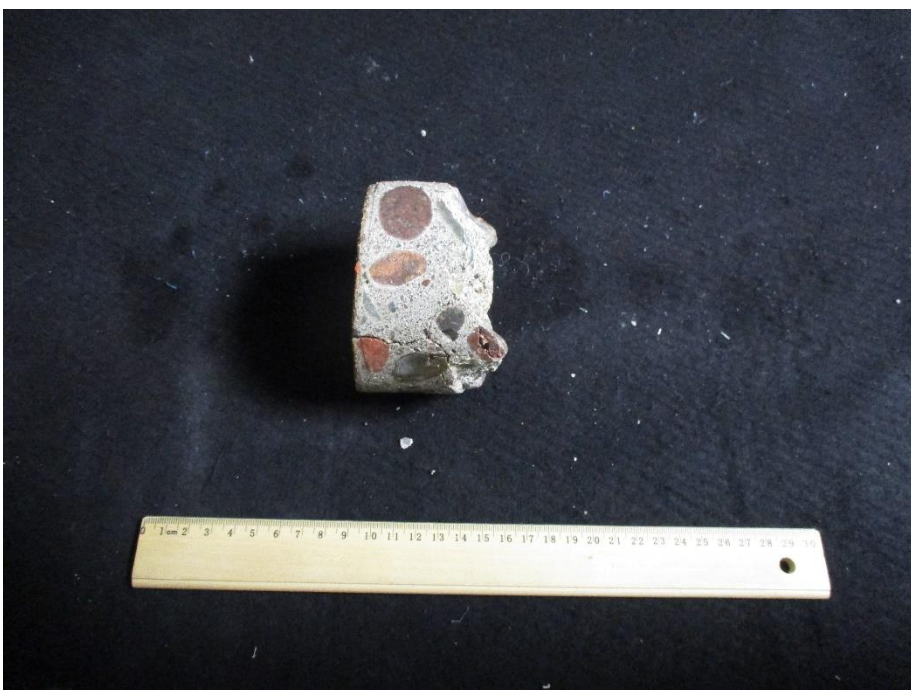
Core 31
Sampled from Swing Pier – Top face



Core 32
Sampled from Swing Pier – Top face

Core 33
Sampled from Swing Pier – Vertical face

Core 33
Sampled from Swing Pier – Vertical face

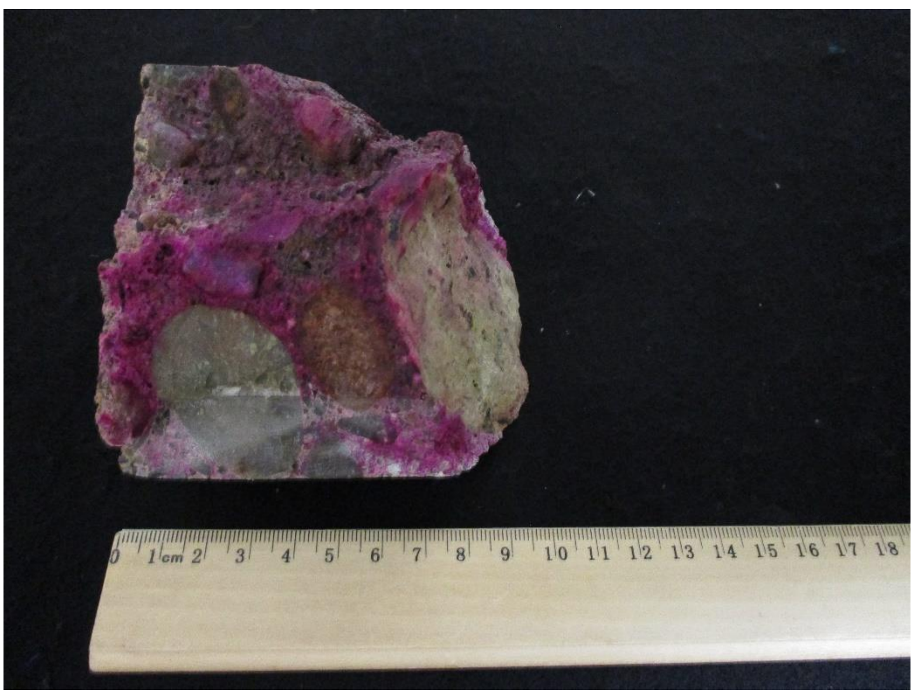


Core 32
Sampled from Swing Pier – Top face

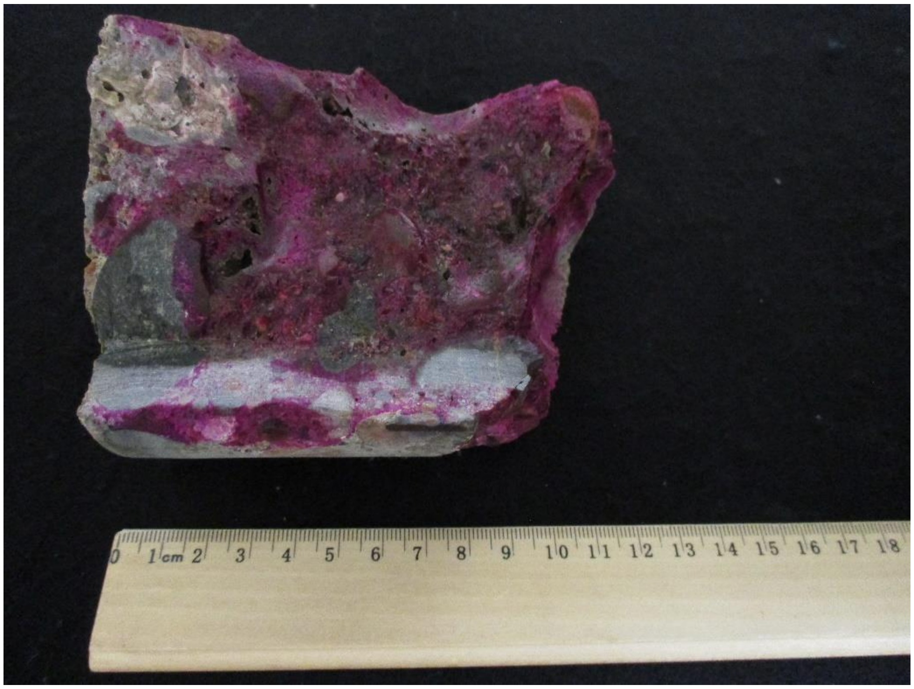
Core 33
Sampled from Swing Pier – Vertical face

Core D5 – Carbonation Depth
Sampled from Bridge Deck

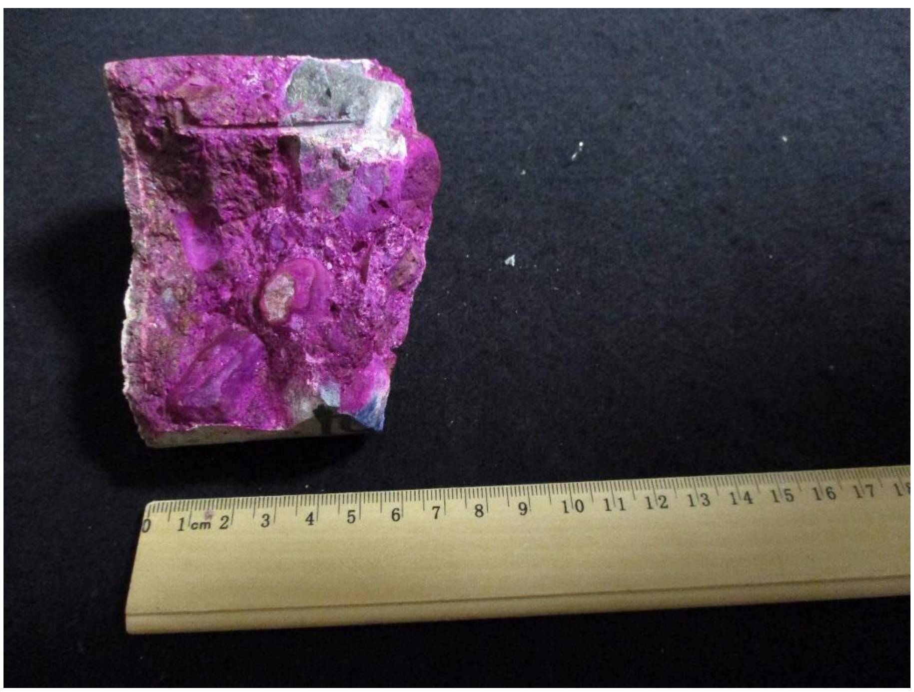
Core D8 – Carbonation Depth
Sampled from Bridge Deck



Core 11 – Carbonation Depth
Sampled from South Abutment – East Wing Wall



Core 28 – Carbonation Depth
Sampled from Pier 1 – Above high-water line



Core 32 – Carbonation Depth
Sampled from Swing Pier – Top face

Compressive Test Results

CBCL

Certified Laboratory
Concrete Testing

Client Name:	NLDTI	**Project No.:**	223049.10
Project Name:	Marystown Bridge (Part A - 39-22PSI)	**Date Tested:**	
Site Sampled:	Marystown Bridge	**Tested by:**	
Date Sampled:			

Compressive Strength - Concrete Drilled Cores: CSA A23.2-14C

Core ID	Length (mm)	Diameter (mm)	Load (lbs)	Mass (g)	L / D	Correction Factor	Uncorrected Compressive Strength (MPa)	Corrected Compressive Strength (MPa)
29	153	95	68400	-	1.61	0.96	42.9	41.3
15	111	95	62900	-	1.17	0.88	39.5	34.8
25	130	95	78430	-	1.37	0.93	49.2	46.0
24	123	95	86680	-	1.29	0.93	54.4	50.7
1	83	95	79400	-	0.87	0.86	49.8	43.0
30	170	95	76630	-	1.79	0.98	48.1	47.2
27	108	95	74940	-	1.14	0.88	47.0	41.3
22	116	95	80900	-	1.22	0.88	50.8	44.9

Notes:	Moisture conditioning:	Dry
	Age of concrete:	Unknown
	Comments:	

Reviewed by: J. Nugent, M.Eng., P.Eng.

Chloride Ion Content

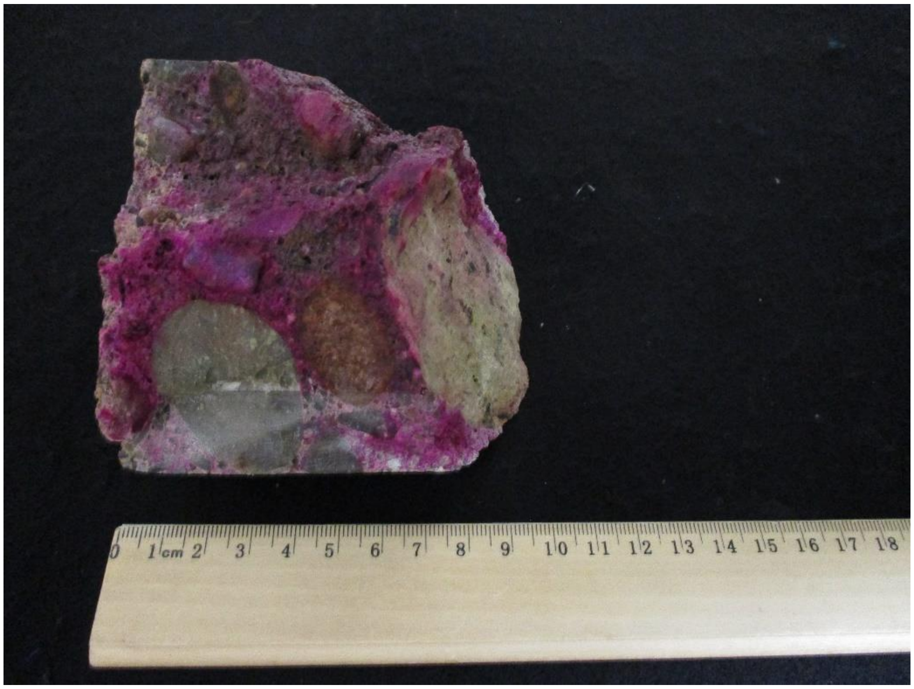
Table C1 - Water-soluble chloride ion concentration (%)

Core ID	Depth from filter vertical face (mm)								
	12.5	25	37.5	50	62.5	75	87.5	100	112.5
2	0.009	0.008	0.007	0.006	0.005	0.003	-	-	-
9	0.005	0.004	0.004	0.005	0.003	0.005	-	-	-
12	0.044	0.030	0.016	0.008	0.005	0.004			
13	0.029	0.016	0.008	0.004	0.005	0.005			
17	0.009	0.005	0.004	0.004	0.009	0.006			
	0.006	0.005	0.009	0.004	0.003	0.004			

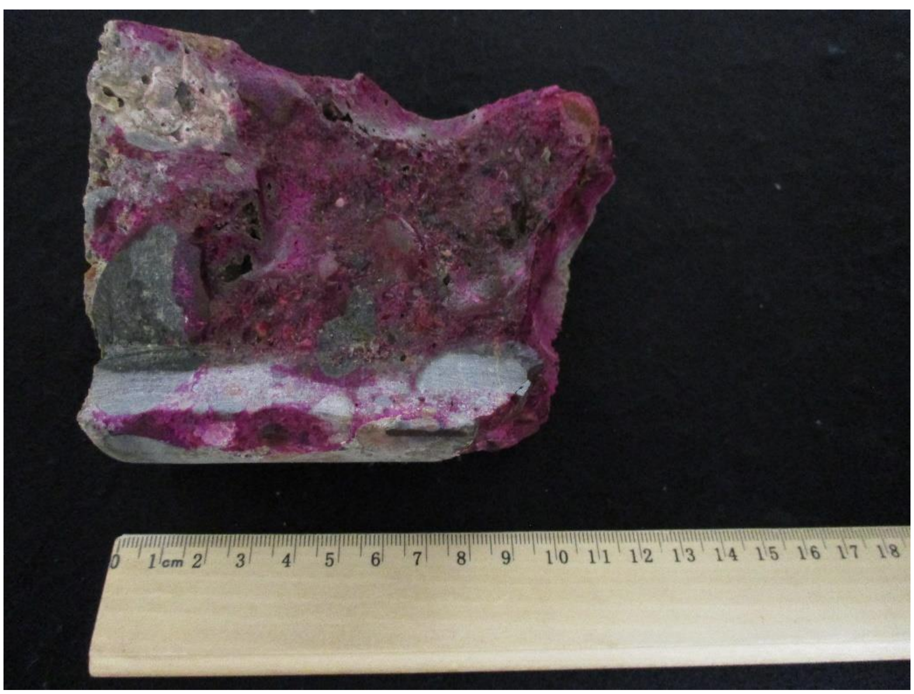
Carbonation Photo Log

Core D5 – Carbonation Depth
Sampled from Bridge Deck

Core D8 – Carbonation Depth
Sampled from Bridge Deck



Core 11 – Carbonation Depth
Sampled from South Abutment – East Wing Wall

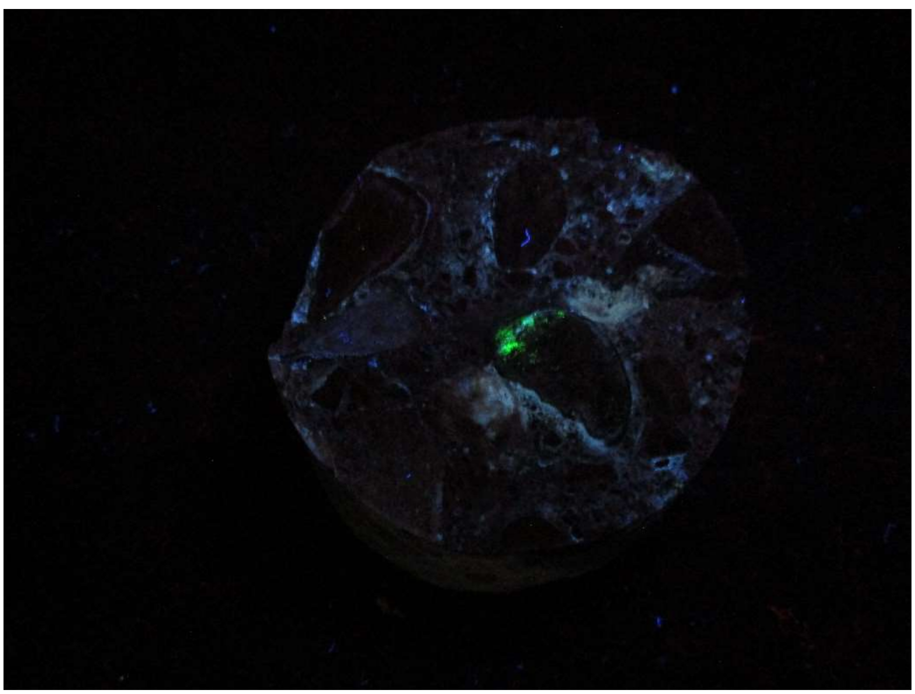


Core 28 – Carbonation Depth
Sampled from Pier 1 – Above high-water line

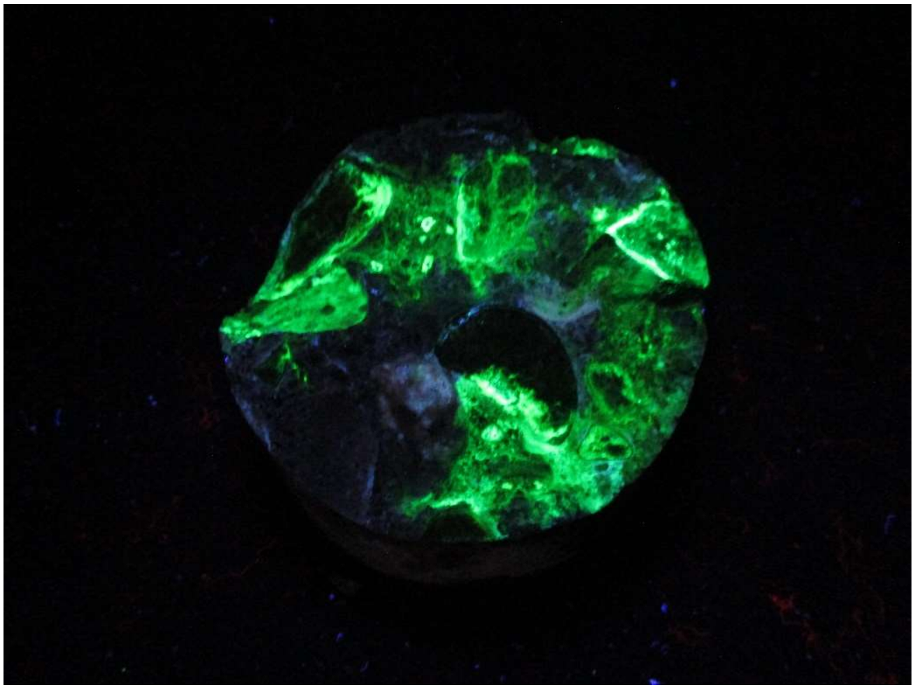
Core 32 – Carbonation Depth
Sampled from Swing Pier – Top face

Gel Fluorescence Photo Log

Core 9
Under Regular Light

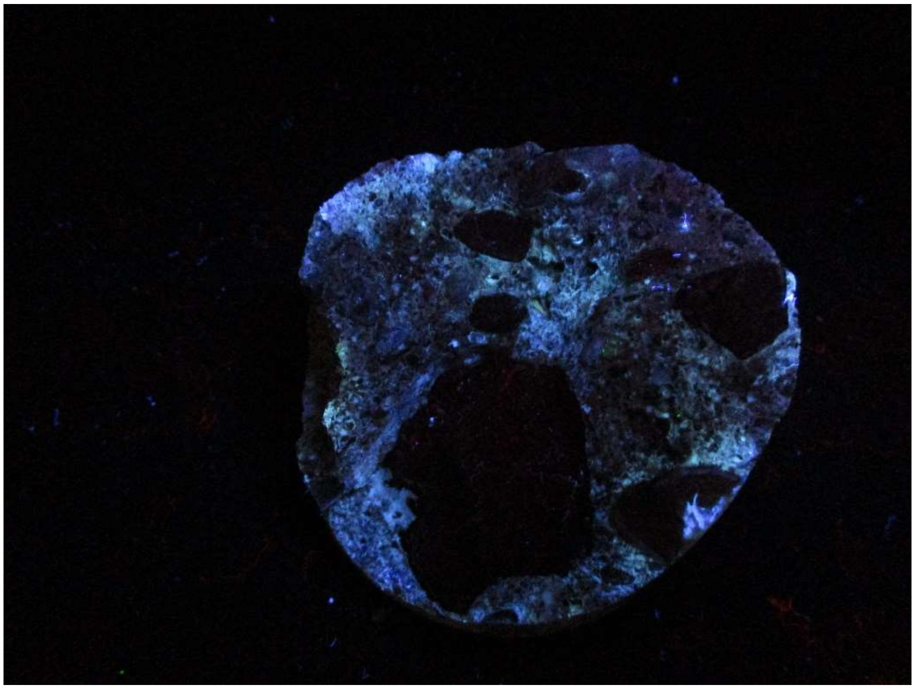


Core 9
Under UV Light

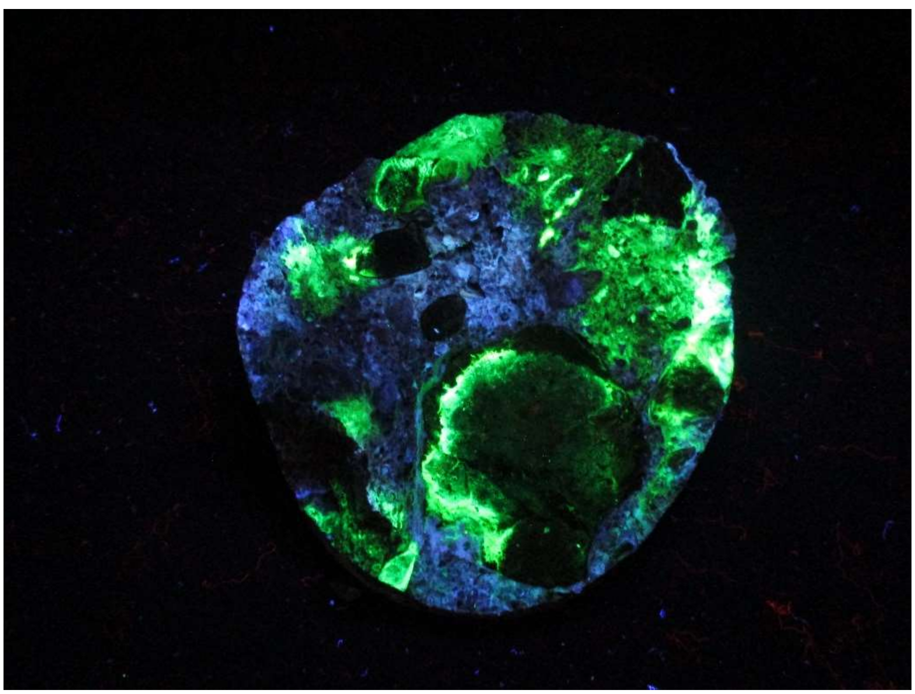


Core 9
Under UV Light following Exposure to Uranyl Acetate Solution

Core 12
Under Regular Light

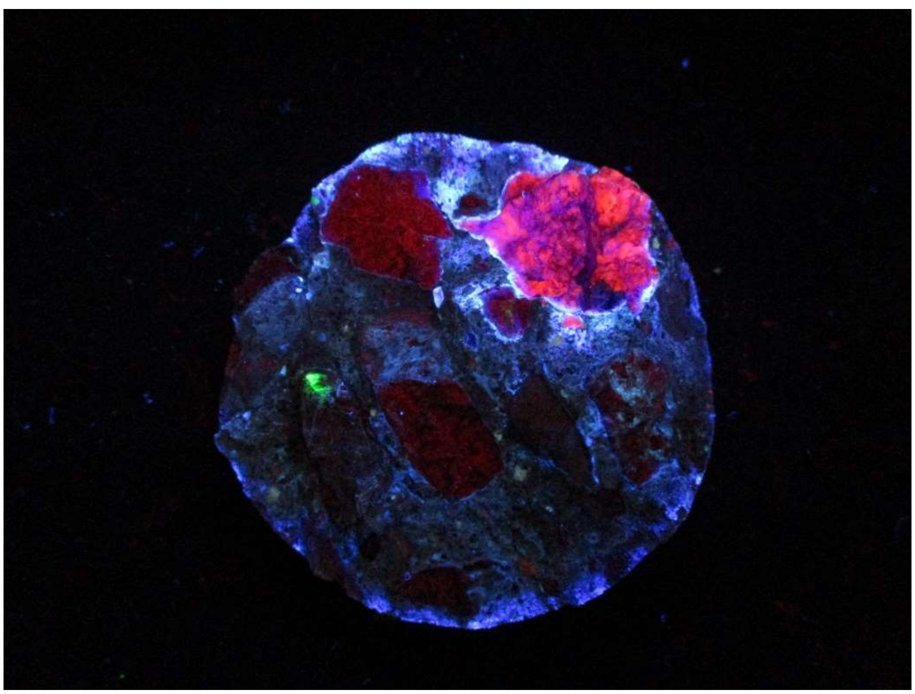


Core 12
Under UV Light

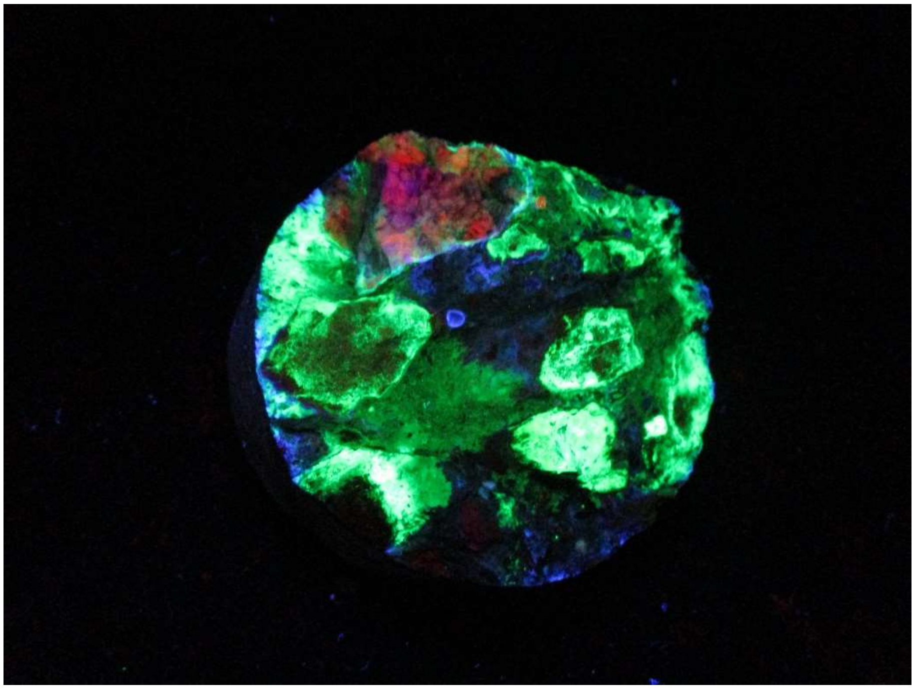


Core 12
Under UV Light following Exposure to Uranyl Acetate

Core 17
Under Regular Light

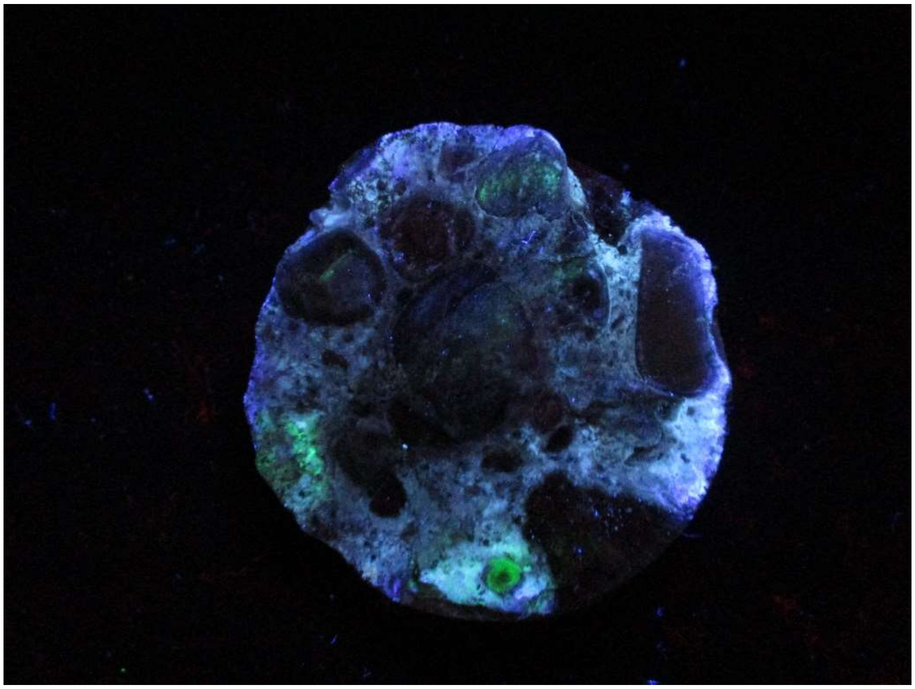


Core 17
Under UV Light

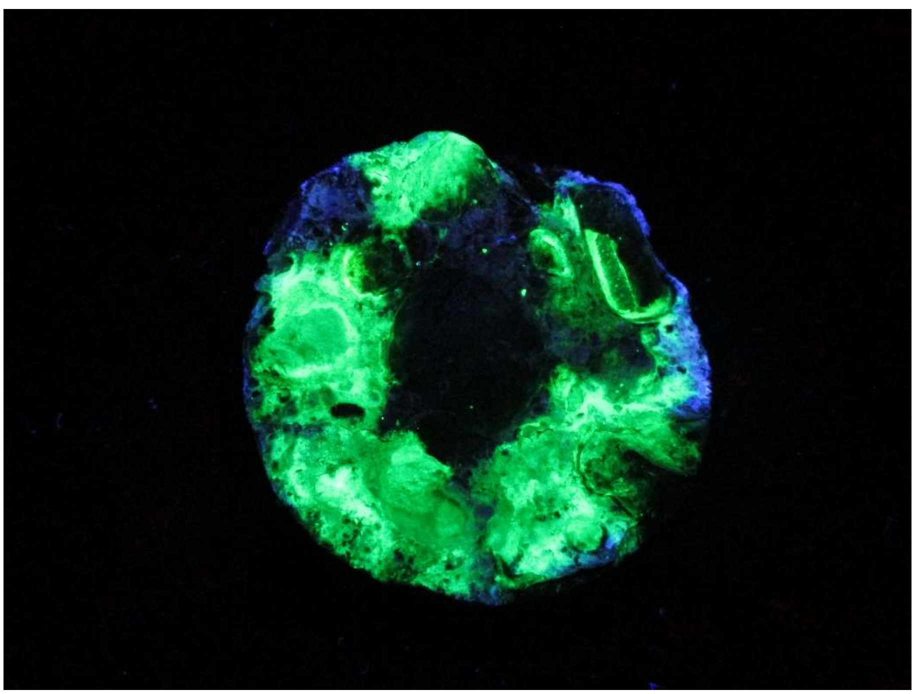


Core 17
Under UV Light following Exposure to Uranyl Acetate

Core 18
Under Regular Light

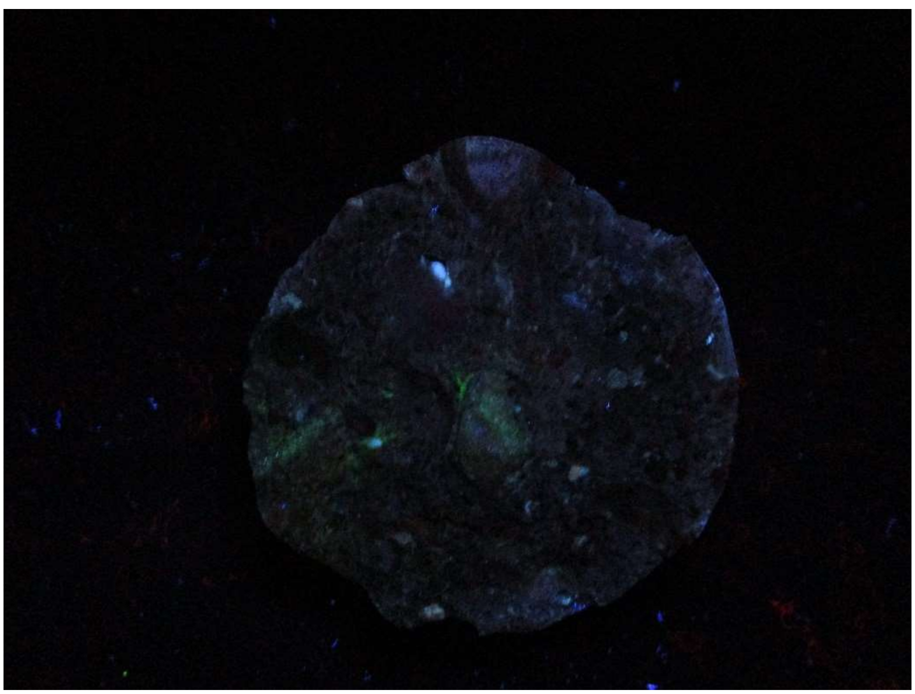


Core 18
Under UV Light

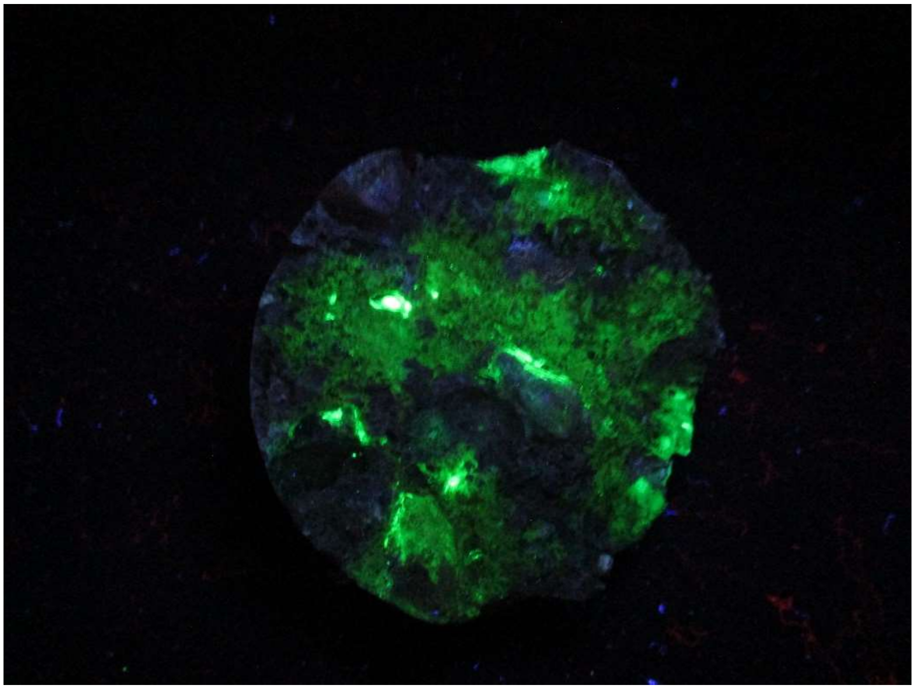


Core 18
Under UV Light following Exposure to Uranyl Acetate

Core 28
Under Regular Light



Core 28
Under UV Light



Core 28
Under UV Light following Exposure to Uranyl Acetate

Air Void Analysis

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO:	223049.1	SAMPLE NO:	223049.10 - 10 - 16
CLIENT:	NLDTI	SAMPLE DATE:	
PROJECT:	Marystown Harbour Bridge	TEST DATE:	25-Nov-22
		TESTED BY:	
		SPECIFICATION:	ASTM C457, Procedure 'B'
ATTENTION:			100x Magnification

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.1
Spacing Factor (mm):	0.37
Traverse Length (mm):	1860.9
Total Stops:	1577
Void Frequency:	0.09
Specific Surface (mm^2/mm^3):	17.36
Paste to Air Ratio:	10.03

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: ______________________ P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10- 10 - 24

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 25-Nov-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.4
Spacing Factor (mm):	1.05
Traverse Length (mm):	1805.4
Total Stops:	1530
Void Frequency:	0.03
Specific Surface (mm^2/mm^3):	5.84
Paste to Air Ratio:	9.31

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 10-3

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 22-Nov-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	6.6
Spacing Factor (mm):	0.12
Traverse Length (mm):	1812.5
Total Stops:	1536
Void Frequency:	0.61
Specific Surface (mm^2/mm^3):	36.59
Paste to Air Ratio:	4.77

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 10-14

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 8-Dec-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	1.0
Spacing Factor (mm):	1.16
Traverse Length (mm):	1851.4
Total Stops:	1569
Void Frequency:	0.02
Specific Surface (mm^2/mm^3):	8.59
Paste to Air Ratio:	28.67

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 - 10 - 16

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 25-Nov-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.1
Spacing Factor (mm):	0.37
Traverse Length (mm):	1860.9
Total Stops:	1577
Void Frequency:	0.09
Specific Surface (mm^2/mm^3):	17.36
Paste to Air Ratio:	10.03

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10- 10 - 24

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 25-Nov-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.4
Spacing Factor (mm):	1.05
Traverse Length (mm):	1805.4
Total Stops:	1530
Void Frequency:	0.03
Specific Surface (mm^2/mm^3):	5.84
Paste to Air Ratio:	9.31

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

[Your Company name here]

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 10-29

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 21-Nov-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	3.5
Spacing Factor (mm):	0.55
Traverse Length (mm):	1774.7
Total Stops:	1504
Void Frequency:	0.08
Specific Surface (mm^2/mm^3):	9.39
Paste to Air Ratio:	6.37

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 40-14

CLIENT: NLDTI

SAMPLE DATE:

PROJECT: Marystown Harbour Bridge

TEST DATE: 9-Dec-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

ATTENTION:

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.2
Spacing Factor (mm):	0.29
Traverse Length (mm):	1834.9
Total Stops:	1555
Void Frequency:	0.15
Specific Surface (mm^2/mm^3):	27.62
Paste to Air Ratio:	17.26

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

PROJECT NO: 223049.1

SAMPLE NO: 223049.10 40-24

CLIENT: NLDTI

PROJECT: Marystown Harbour Bridge

ATTENTION:

SAMPLE DATE:

TEST DATE: 8-Dec-22

TESTED BY:

SPECIFICATION: ASTM C457, Procedure 'B'

100x Magnification

Concrete Properties	
Supplier:	
Mix Code:	
Specified 28-day Strength (MPa):	
Specified Air Content (%):	
Cement Type:	
Aggregate Size (mm):	
Specified Slump (mm):	

Air-Void Analysis	
Hardened Air Content (%):	2.5
Spacing Factor (mm):	0.13
Traverse Length (mm):	1799.5
Total Stops:	1525
Void Frequency:	0.38
Specific Surface (mm^2/mm^3):	60.21
Paste to Air Ratio:	14.95

Plastic Concrete Properties	
Concrete Temp (°C):	
Air Content (%):	
Initial Slump (mm):	
Final Slump (mm):	

Compressive Strength	
7 Day (MPa):	
28 Day (MPa):	
28 Day (MPa):	
56 Day (MPa):	

REMARKS:

REVIEWED BY: P.Eng.

Petrographic & Damage Condition Rating Report

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February 3, 2023

Draft - Study of concrete from Marystown Narrows Bridge, Newfoundland

Background

This bridge is reported to have been constructed in 1957. The concrete is undergoing a condition survey. As part of this study, a petrographic examination of the concrete was undertaken.

Samples

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

Table 1: Summary of core identification and dimensions

Core identification	Diameter and length of polished face	Notes
Core 10-29 PE	94 x 280 mm	Core split in half by diamond sawing. From the south (or west) abutment foundation. One end outer surface was a formed surface within the structure (see figure1).

Sample Preparation

The core had been cut length ways with a diamond saw. Each half core had been polished with a variety of abrasives to produce a flat polished face. Photographs of the polished core are shown in figures 1 to 3. To facilitate further polishing and examination one half core was split into two halves by further diamond sawing.

Techniques

The polished surfaces were examined under a binocular microscope in reflected light and a Damage Rating Index (DRI) established for the core (see Annex to this report). Examination was done at a magnification of 16x. The results are shown in table 2.

Following examination of the polished surface, the 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 core, crushed and mounted on glass slides as powders. The powders were immersed in refractive index oils of various values. 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 an 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.

Observations of Concrete condition

Petrographic Examination of Hardened Concrete	
Project name	Concrete from Marystown Narrows Bridge
Sample Numbers	Core 10-29 PE
Sample type and size	Polished section of diamond drill core 94 x 280 mm
Concrete general condition	Moderately strong requiring one or two hammer blows to fracture a half core. Fractures on pre-existing cracks, bond of coarse aggregate to mortar was poor with many aggregate/paste sockets. Carbonation – from the formed surface no carbonation. Presumably this was an interior surface not exposed to the atmosphere. Damage Rating Index: = 431. The part of the core close to the formed surface and shown in figure 1 showed significantly less ASR damage than the portion shown in figure 2.
Max. aggregate size	70 mm.
Aggregate grading	Satisfactory. Use of particles larger than 25 mm is unusual by modern standards.
Coarse aggregate	Composed of rounded uncrushed gravel with generally poor bond to mortar with sockets in the mortar where coarse aggregate was removed on hitting with a hammer. These sockets often had a lining of minute thin hexagonal crystals of portlandite ($Ca[OH]2$) which is not abnormal. Coarse aggregate content: About 46% Composed of a mixture of siliceous volcanic rocks varying in colour from grayish red, grayish brown to light grey and medium light grey depending on individual composition. Volcanic rocks composed of mixtures of rock fragments, quartz and feldspar sand size grains in a very fine grained matrix that was probably originally a volcanic glass but has recrystallized. The general term for such rock is a tuff. A tuff is a term used for pyroclastic volcanic rock laid down as a sediment either above (subaerially) or below water. A pyroclastic rock is formed by accumulation of fragments from a volcanic explosion from a vent. The tuffs were characterized by poor internal particle grading (sorting) being composed of a mixture of large and small fragments in a glassy groundmass. The volcanic rocks varied from rhyolitic to dacitic in composition. There were a few dark rims on the polished surface but these may have been due to weathering in the gravel deposit and cannot be reliably described as 'reaction rims'. Trace amounts of unoxidized pyrite, often cubic in shape, were found in about 16 individual particles. There were trace amounts (< 5%) of very light grey quartz rich sandstone and pale red coloured granite. There were obvious signs of ASR associated with the tuff. The majority of

	tuffaceous particles were cracked and the cracks lined with ASR products. On the fracture surfaces in the coarse aggregate particles there were thin dark rims of glassy alkali-silica gel with white crystalline deposits in the interior (figure 5). The coarse aggregate particles were well graded, generally sound, unweathered and judged physically suitable as a concrete aggregate when originally used but many are now partly fractured due to alkali-silica reaction.
Fine aggregate	Natural sand with poor bond to paste with few fractured particles on broken surfaces when hit with a hammer. Rounded to sub-angular natural sand particles. Composed of siliceous volcanic rocks similar to coarse aggregate in composition. Few quartz particles were observed. The fine aggregates were judged to be well graded but generally slightly coarse with an absence of many fine particles, generally consisting of sound and strong particles.
Cement paste	Dull appearance, w/c >0.50? Colour very light grey.
Air voids	Not air-entrained (about 2 to 4 % voids). Voids ranged from about 0.5 to 3 mm in diameter – partly entrapped air? Very poor spacing factor.
Secondary minerals in air voids	Alkali-silica gel present as clear glassy (isotropic) brittle deposits (old?) was found lining the outer part of air voids usually close to coarse aggregate particles affected (cracked) by ASR. Usually there was a soft, very fine crystalline white powder as a reaction product in the interior of these voids. Refractive index was in range from about 1.49 -1.50 for both products. The white material gave 1st order black and grey colours, very fine grained with crystals < 5 μm. This apparently crystalline reaction product may be a calcium silicate hydrate probably with some potassium in the structure and is often found in concrete affected by ASR and is considered a product generated as the reaction proceeds (Katayama 2012). The majority of air voids contained ettringite as a thin silky looking lining. The ettringite ($3CaO.Al_2O_3.3CaSO_4.31H_2O$) occurred as white needles up to about 0.15 mm long. Identification was based on a refractive index of slightly less than 1.47 and parallel extinction of needle shaped crystals with the characteristic low birefringence. The amount observed was considered unexceptional and considered normal for concrete of this age in a relatively dry environment? Ettringite needles were not found growing perpendicular to the void walls but as mats parallel to the void walls. Concrete that has been in a wet environment and had a high moisture content usually has the needles growing perpendicular to the void walls. A special examination was made for the presence of thaumasite ($CaSiO_3.CaCO_3.CaSO_4.15H_2O$). This is a material very similar in appearance to ettringite and responsible for damaging sulphate attack and found in concrete exposed to moisture and stored at relatively low temperatures. Refractive index determination showed that the ettringite observed had a refractive index of about 1.47 confirming the identification. Thaumasite has a refractive index of about 1.50.

Embedded materials	No steel present but one small wood fragment observed.
Cracks	The mortar and coarse aggregates were micro-cracked as shown by the DRI observations. These cracks were narrow, usually < 10 to 20 μm wide, and sometimes lined with white secondary material. There were some open cracks or seams in the coarse aggregates that appeared to predate concrete mixing and had been caused by weathering in the gravel deposit. Their presence results in a higher DRI than would normally be the case. Other cracks appeared to be due to expansion caused to alkali-silica reaction.
Secondary minerals in cracks	At the edge of fractured coarse aggregate particles there were dark deposits of probable alkali-silica gel. The white coating on crack surfaces in the coarse aggregates (figure 5) was composed of very fine grained, soft, crystalline white coloured material < 5 μm in crystal size with very low birefringence. The refractive index was about 1.49. This was not an alkali-silica gel but a reaction product with a crystalline nature. The mineral is probably a calcium silicate hydrate. It is possible there is sodium and potassium in the structure making it slightly different from natural minerals. Note: Further examination using a scanning electron microscope would be necessary to determine composition of the white mineral coating fracture surfaces. This observation of white powder on crack surfaces of coarse aggregate affected by ASR is common with alkali-silica reactive rocks found in Atlantic Canada. There is a fuller description of the possible mineralogy in Katayama and Futagawa (1989) and in Katayama (2012).
Summary	Moderate strength, well proportioned, non air-entrained concrete. Strong evidence of alkali-silica reaction.

Discussion

The concrete in all cores was reasonably well proportioned but of large maximum particle size (70 mm), well consolidated but of moderate strength due in part to poor aggregate/mortar bond. The concrete was not air-entrained.

The geological map of the area in the vicinity of Marystown shows the majority of the bedrock is assigned to the Marystown and other named groups. These are a Neoproterozoic stratigraphic group (approx 620 to 550 Ma) of predominantly volcanic sediments with some sandstone units. These were probably subaerially and submarine deposited ash-flow tuffs of rhyolitic composition. There are also outcrops of granite found due north of Marystown on the west side of the Burin Peninsular. The rock types found in the concrete correspond to this description and must have been taken from a relatively local gravel deposit.

There were deposits of the product of alkali-silica reaction (ASR) both as alkali-silica gel and a crystalline material tentatively identified as a calcium silicate hydrate in air voids and crack surfaces in the mortar and aggregate. The amount of ASR product observed was judged to be moderate.

At the time of construction, in 1957, knowledge about the occurrence of alkali-silica reaction in Newfoundland was non-existent and the locally available cement had a high alkali content (North Star Cement).

The Damage Rating Index (DRI) results shown in Table 2 show a value of 431. This value indicates that the concrete has been damaged by alkali-silica reaction. There was cracking of both the coarse aggregate and mortar indicating expansion of the concrete. A graph of DRI versus laboratory expansion generated by Sanchez *et al* (2017) is shown in the annex. Using this it is possible to estimate the amount of likely expansion of the concrete. Based on a DRI of 431 figure 4 of the annex indicates that expansion is likely to be in the range from about 0.05% to about 0.15% which is relatively minor given the age of the concrete (60 years). It should be noted that the data generated by Sanchez is based on laboratory specimens and not on long term exposure of concrete in the field where freezing and thawing takes place.

The ettringite observed was not unusual. The amount of ettringite was relatively low and suggests this concrete sample may have been relatively dry.

Conclusions

1. The concrete in was well proportioned and well consolidated, non air-entrained and of moderate strength.
2. The Damage Rating Index was 431. The coarse aggregate and mortar showed cracking with deposits of alkali-silica gel in some air voids adjacent to coarse aggregate particles and deposits of a probable calcium silicate hydrate on fracture surfaces. There has probably been some swelling / expansion of the concrete.
3. The rock types responsible for the alkali-silica reaction were volcanic tuffs. These rocks types are well known to be responsible for damaging alkali-silica reaction in Newfoundland (Bragg, 2000).
4. Observations of ettringite in air voids are common in most concrete and the occurrence in this concrete was not judged to be unusual.

Chris Rogers.

Chris Rogers

References

Bragg, D., *Alkali-aggregate reactivity in Newfoundland, Canada,* Canadian Journal of Civil Engineering, vol.27, pp. 193-203, 2000.

Katayama, T., and T. Futagawa, *Alkali-aggregate reaction in New Brunswick, Eastern Canada - Petrographic diagnosis and deterioration,* Proceedings 8th Internat. Conf. on AAR, Kyoto, Japan, pp. 531-536, 1989.

Katayama, T., *ASR gels and their crystalline phases in concrete - Universal products in alkali-silica, alkali-silicate and alkali-carbonate reactions,* 14th International Conference on Alkali-Aggregate Reaction, Austin, Texas, 2012, paper 030411 Kata 03.

Mills, Andrea, J. et al, 2020: Lithogeochemical, isotopic, and U–Pb (zircon) age constraints on arc to rift magmatism, northwestern and central Avalon Terrane, Newfoundland, Canada: implications for local lithostratigraphy: https://cdnsciencepub.com/doi/10.1139/cjes-2019-0196

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

Sample Information: Marystown Narrows Bridge, Core 10-29 PE, January 23, 2023.

FEATURE	TOTAL COUNT	FACTOR	Contribution to DRI
Cracks in coarse aggregate	143	0.25	35.75
Open crack in coarse aggregate	28	2	56
Crack with reaction product in coarse aggregate	245	2	490
Disaggregated/corroded aggregate particle	0	2	0
Coarse aggregate debonded	15	3	45
Crack in mortar	19	3	57
Crack with reaction product in mortar matrix	91	3	273
Air void containing AS gel	6	No value in DRI	-
Reaction rim	0	No value in DRI	-
Total (Uncorrected Damage Rating Index)			956.75
Total area of sample viewed 19 x 13 - 12 = 222 cm^2			222 cm^2
DRI, normalized to 100 cm^2			431

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

.

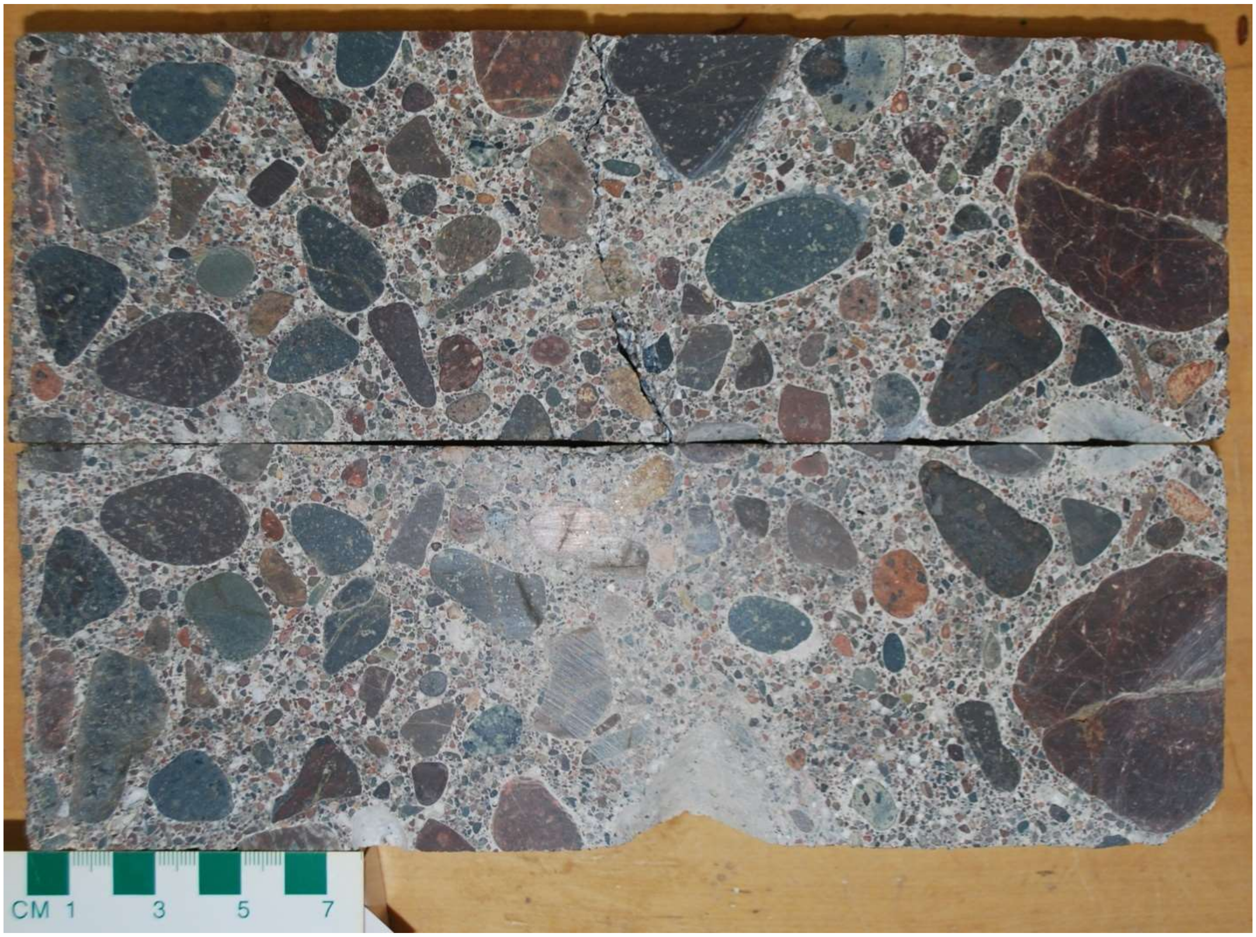


Figure 1: Core 10-29 PE as received. Formed concrete surface on left edge. Total length about 280 mm.

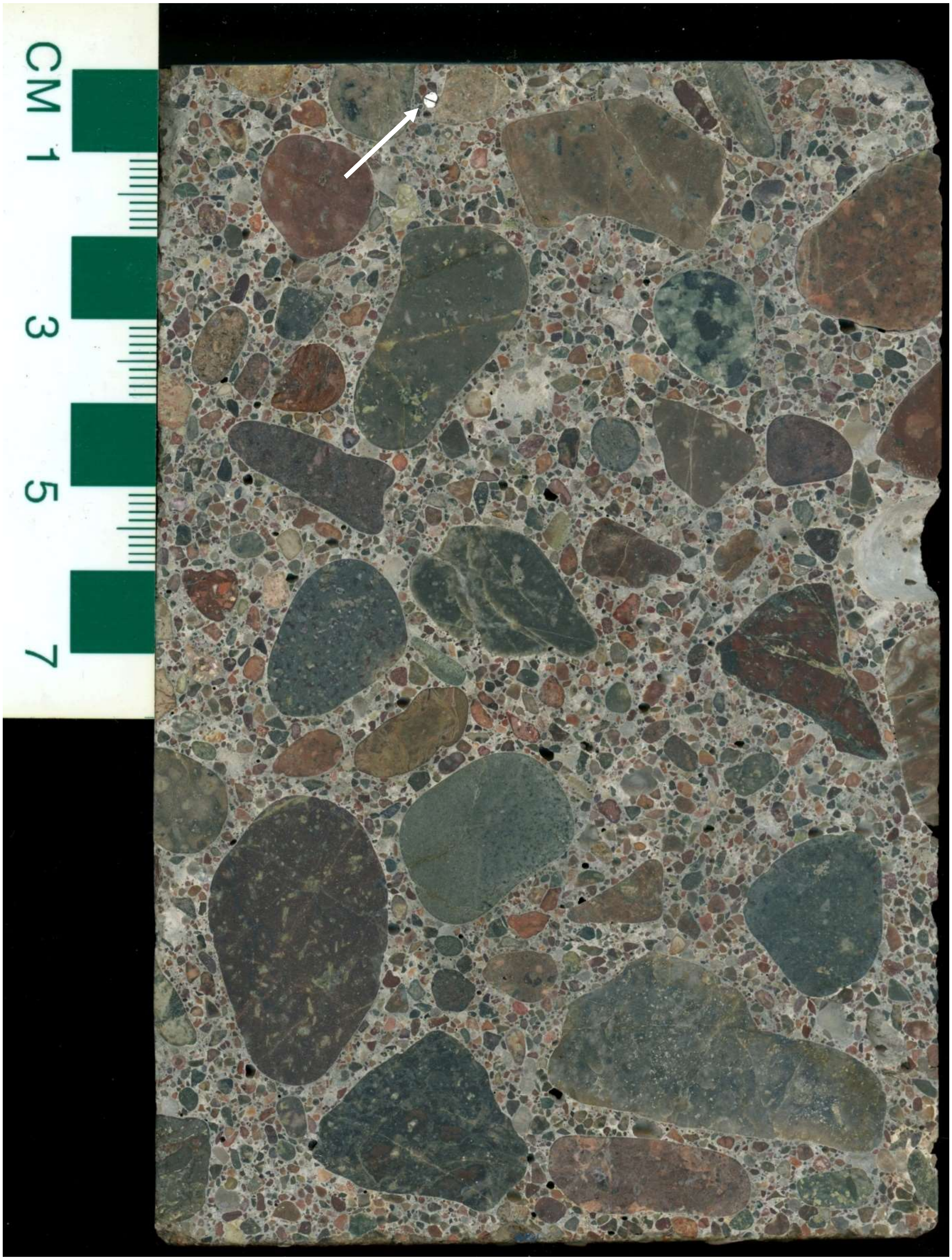


Figure 2: Core 10-29 PE part of core. White alkali-silica gel filled air void highlighted by arrow. Formed surface is on bottom of core image.

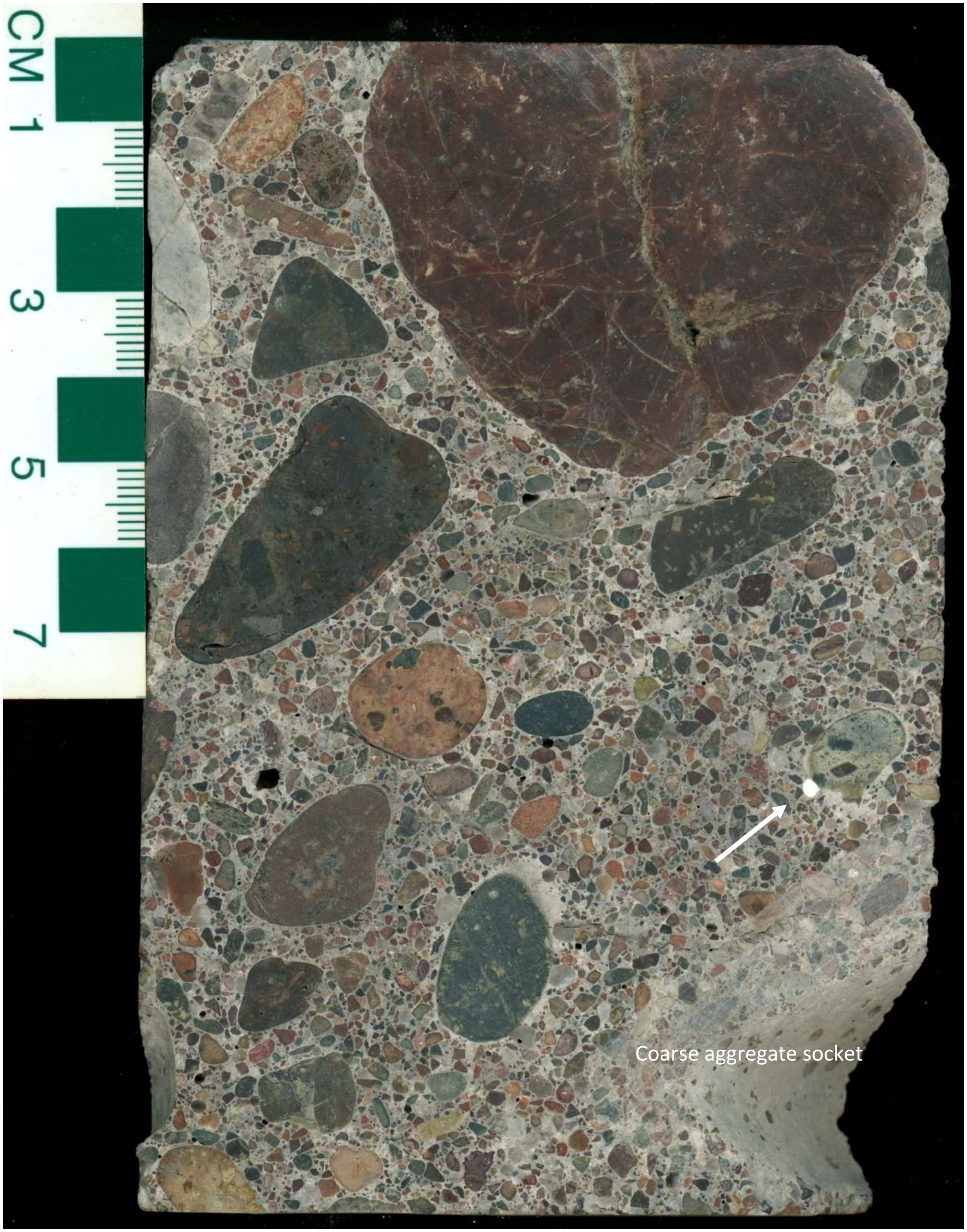


Figure 3: Core 10-29 PE part of core. White alkali-silica gel filled air void highlighted by arrow.

Figure 4: Detail of lower left area shown in figure 2 showing feldspar and rock fragments in pyroclastic tuff.

Figure 5: Fracture surface showing dark rim of alkali-silica gel on edge of coarse aggregate particle with white soft AS reaction product in interior of fracture surface through the particle. Field of view = 35mm.

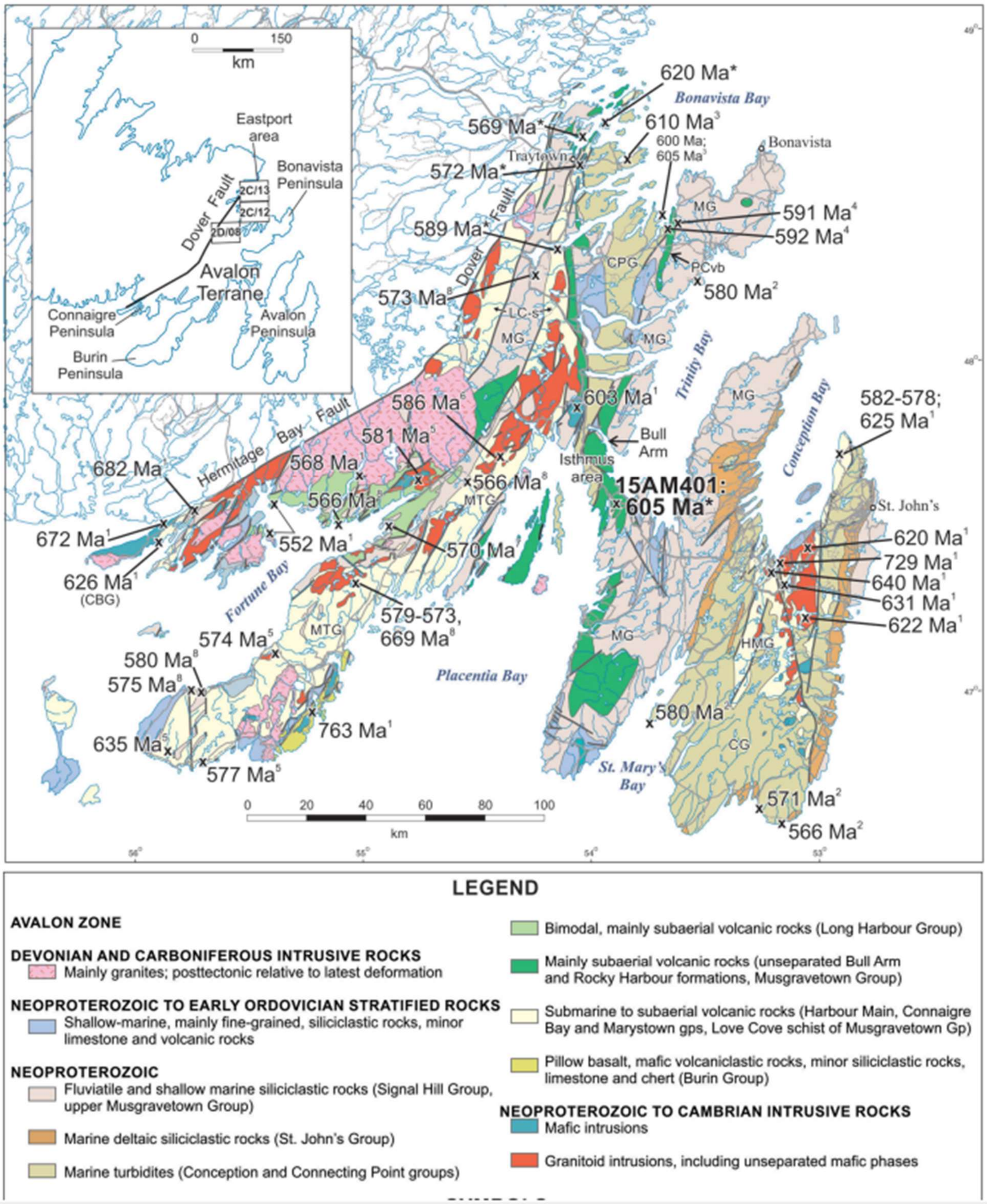


Figure 6: Simplified geological map of the Avalon Terrane in Newfoundland from Mills et al 2020.

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.

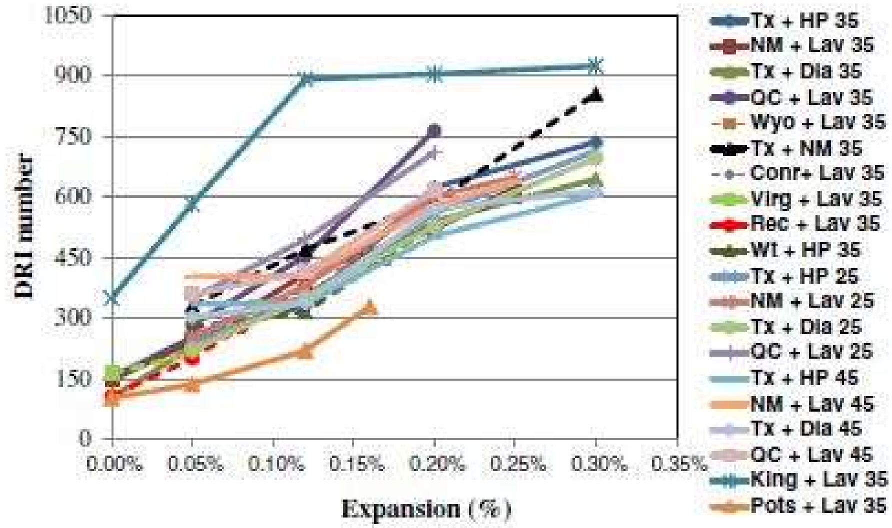


Fig. 4. DRI number for all mixtures analyzed in this study.

Figure 4, from Sanchez et al, 2017.

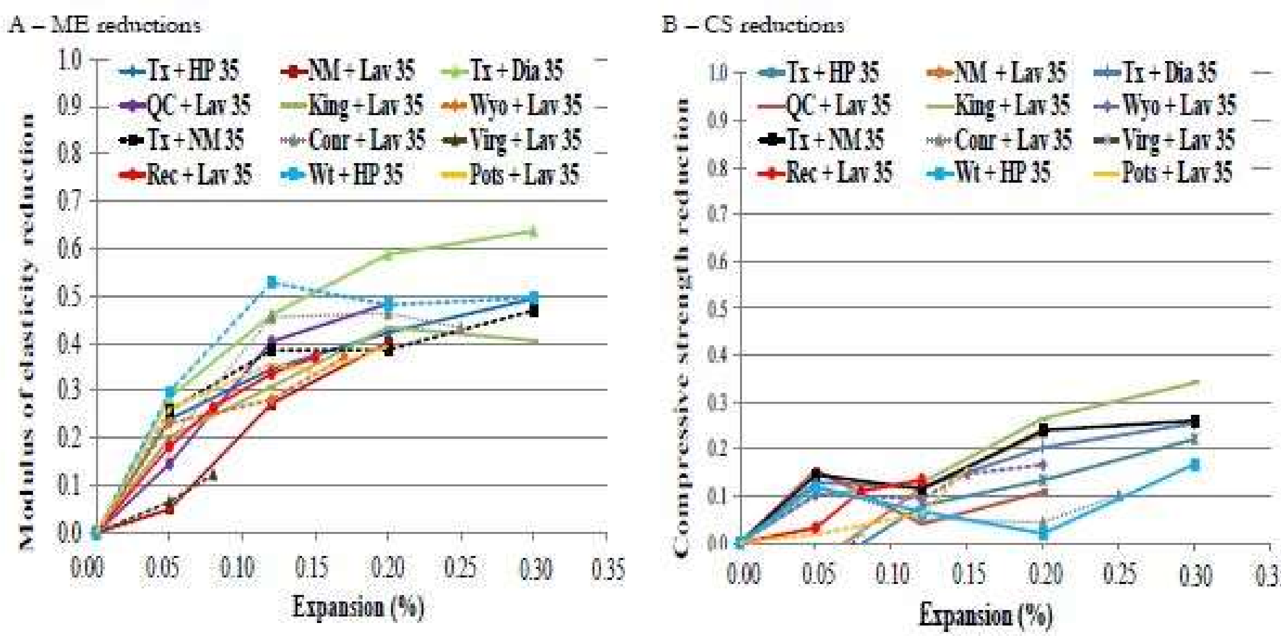


FIGURE 3: Reductions in the modulus of elasticity (ME) (A) and compressive strength (CS) (B) as a function of ASR expansion, for 35 MPa concretes incorporating a variety of reactive aggregates.

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.

APPENDIX J

Magnetic Particle Testing Report

Acuren Group Inc.
2 - 240 Taiganova Crescent
Fort McMurray, AB, Canada T9K 0T4
www.acuren.com
Phone: 780.790.1776
Fax: 780.790.9061

A Higher Level of Reliability

NONDESTRUCTIVE EXAMINATION

CLIENT: CBCL Ltd.

DATE: Sept 29, 2022
ACUREN JOB #: 164-J031765
REPORT #: MT-AG-092922-0058
CONTRACT/PO: N/A WO: N/A

ATTENTION: Mitch Warren
WORK LOCATION: Marystown, NL
PROJECT: Marystown Harbour Bridge Inspection
ITEM(S) EXAMINED: Splice Plates, 3 on the continuous, 1 on the swing stage

TAG #: Canning Bridge MATERIAL: CS THICKNESS: N/A
SCOPE: Perform a Wet Visible MT inspection on requested areas of splice plates (4)
TYPE OF INSPECTION: Magnetic Particle

TEST DETAILS:

ACCEPTANCE STANDARD: Client Info REV DATE N/A
PROCEDURE/TECHNIQUE: CAN-MT-14P001 REVISION: REV 17

TYPE: Wet Visible	METHOD: Yoke
PARTICLE BRAND: Magnaflux PRODUCT NO.: 7HF	CURRENT: AC MT INSTRUMENT: Parker B-300
PARTICLE COLOUR: Black	MT INSTRUMENT S/N: 29899 CAL DUE: 13/12/22
SUSPENSION: Oil	LIFT CHECK BEFORE USE: Yes LIFT WEIGHT S/N: MT1941
CONTRAST PAINT: maganaflux PRODUCT NO.: WCP2	LIGHTING EQUIPMENT: Flashlight
MAG TIME (SECONDS): 5-10 DEMAG REQUIRED?: No	BLACKLIGHT MAKE: N/A S/N: N/A
TECHNIQUE DEMONSTRATED OVER A PAINTED SURFACE?: No	LIGHT METER S/N: 2038863 CAL DUE: Jan 12 2023
	LIGHT INTENSITY: > 100fc (1076 Lux)
BATCH NOS. (WHEN REQUIRED): PARTICLES: NA	SUSPENSION: NA CONTRAST PAINT: NA
TEST SURFACE CONDITION: As Grinded	TEST SURFACE TEMPERATURE: 16°C

RESULTS:

As requested, a black on white magnetic particle examination was performed on splice plates (three on the continuous, and 1 on the swing stage), coating was removed and reapplied after inspection. See following pages for photos.

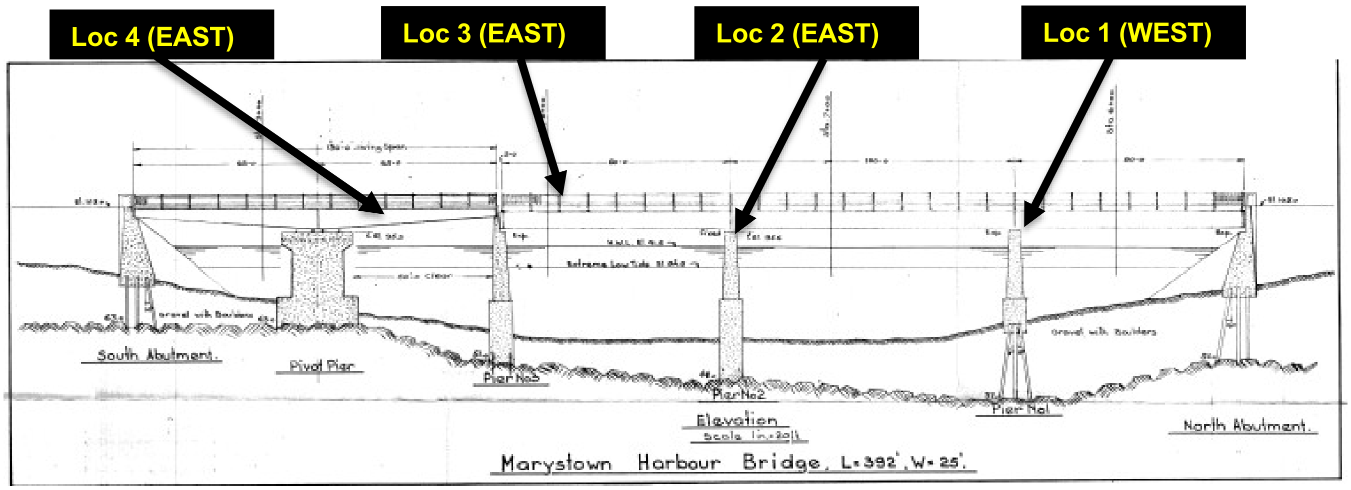
No relevant indications found at time of examination. Acceptable to code.

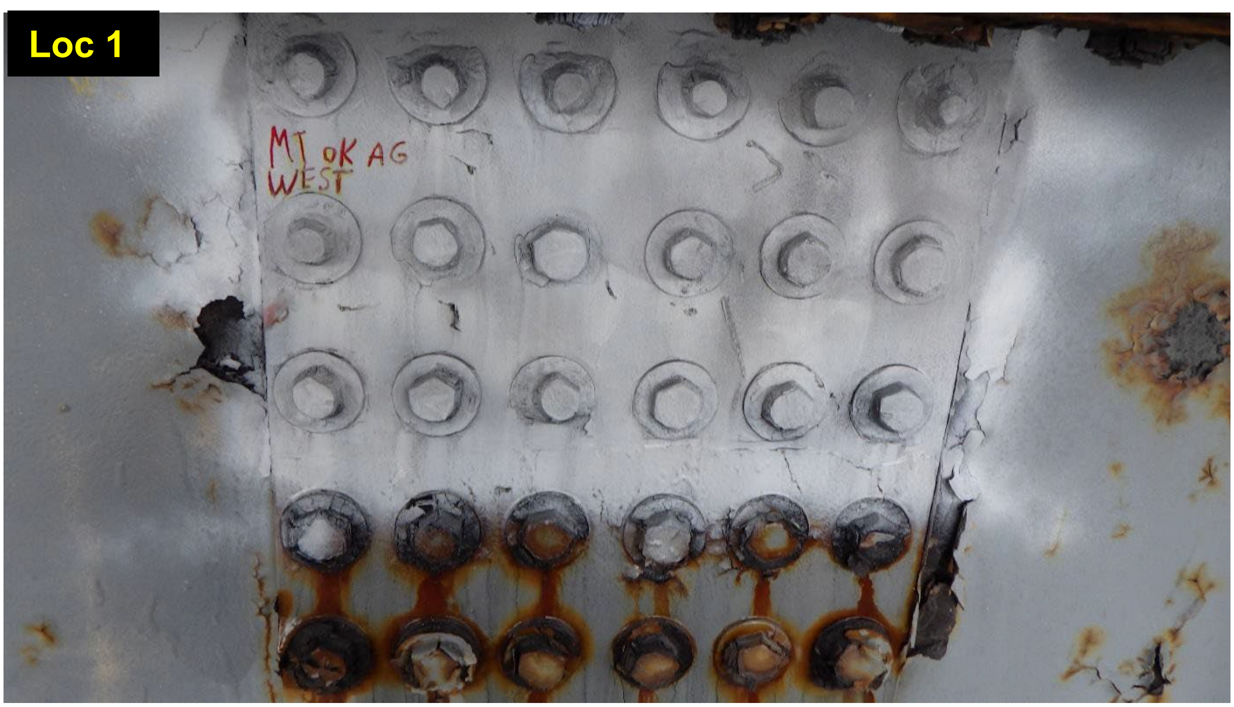
CLIENT: ___
CLIENT PRINTED NAME
CLIENT SIGNATURE
ACCEPTED & ACKNOWLEDGED BY

DTR NO.: N/A

ACUREN TECHNICIAN: Andrew Goodyear
1st Technician
CGSB MT2, UT1 #22706
2nd Technician

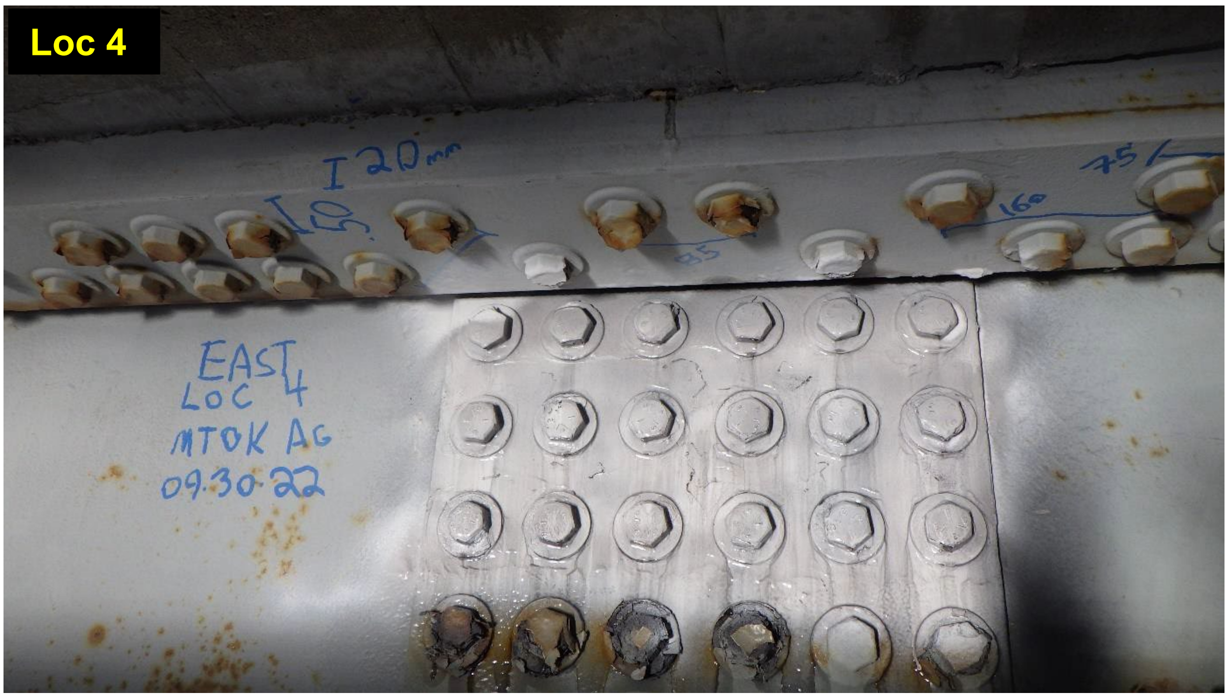
REVIEWER: Jana Matthews
11/08/22


Loc 4 (EAST)
Loc 3 (EAST)
Loc 2 (EAST)
Loc 1 (WEST)
South Abutment.
Pivot Pier
Pier No3
Pier No2
Pier No1
North Abutment.
Elevation
Marystown Harbour Bridge, L=392', W=25'.


Loc 1
MT OK AG
WEST


Loc 2
Loc2 EAST MT OK AG 09-29-22


Loc 3
Loc 3
EAST
MT OK AG
09-29-22


Loc 4
EAST
LOC 4
MTOK AG
09.30.22

APPENDIX K

Hazardous Material Report

ALL-TECH
ENVIRONMENTAL SERVICES LIMITED

170B Roe Avenue
Gander, NL

Bus: 709.754.4146
Fax: 709.754.4194
Email: kwall@toalltech.com

November 1st, 2021

CBCL Limited
187 Kenmount Road
St. John's, NL
A1B 3P9

ATTN: Todd Puddicome

RE: Arsenic, Lead, and Mercury Results – Marystown Harbour Bridge, NL

On October 14, 2022, four (4) suspect paint samples were dropped off to All-Tech Environmental Services office in Mount Pearl from the Marystown Harbour Bridge in Marystown, NL. These samples were forwarded to EMSL Canada Inc. in Mississauga, ON and underwent subsequent laboratory analysis to determine the arsenic, lead, and mercury concentrations. Listed in Table 1.0 are the results of this testing.

Table 1.0
Hazardous Materials Content Results
Marystown Harbour Bridge
Marystown, NL

Sample ID	Arsenic Content		Lead Content		Mercury Content	
	Concentration (mg/kg)	Guidelines (mg/kg)	Concentration (mg/kg)	Provincial Guidelines (mg/kg)	Concentration (mg/kg)	Provincial Guidelines (mg/kg)
NL10455-01	None Detected	12	**9100**	**600**	0.068	10
NL10455-02	None Detected	12	**19000**	**600**	0.088	10
NL10455-03	None Detected	12	**90000**	**600**	None Detected	10
NL10455-04	None Detected	12	**69000**	**600**	0.057	10

Details to be noted:

There are currently no guidance documents for working with arsenic in Newfoundland. For acceptance into a Newfoundland and Labrador landfill, concentration of arsenic must be below the CCME Canadian Soil Quality guidelines for industrial land use (12mg/Kg) or pass the Toxicity Characteristic Leaching Procedure U.S 1311 standard test for leachability for the parameters listed

170B Roe Avenue
Gander, NL
Bus: 709.754.4146
Fax: 709.754.4194
Email: kwall@toalltech.com

in Schedule II Leachate Test, CEPA proposed Regulation, 2002 (2.5 mg/L).

The Treasury Board of Canada's *Handbook of Occupational Safety and Health* has several sections which apply to lead. Volume 12, Chapter 3, TB STD 3-2, Dangerous Substances Safety Standards has regulations for the control of airborne contaminants which also apply to lead. The standards indicate that airborne contaminants *"do not exceed the threshold limit value recommended by the American Conference of Governmental Industrial Hygienists in its pamphlet "Threshold Limit Values for Chemical Substances and Physical Agents, 1998."* At this point in time, the ACGIH have set the TLV levels for airborne concentrations of airborne lead at 0.05 mg/m^3. The Newfoundland and Labrador Occupational Health and Safety Regulations (RSNL1990 CHAPTER O-3) Section 25, 11A states:

"The employer shall ensure that

> *(a) atmosphere contamination of the workplace by chemical substances is kept as low as is reasonably practicable and in the case of the substances for which a threshold limit value is currently established by the ACGIH that threshold value shall not be exceeded"*

These limits represent conditions under which it is believed that nearly all workers can be repeatedly exposed day after day, without adverse health effects. Newfoundland & Labrador guidelines have a set limit of 600mg/kg lead by weight (0.06% wt) of paint to be classified as Lead Based Paint.

There are mercury guidance documents for working with mercury in Newfoundland. Therefore, as a screening tool, paint concentrations were compared to the surface coating material regulations SOR/2005. The Surface Coating Regulation SOR/2005 made under the *Canadian Hazardous Product Act* (CEPA) considers paint mercury-based if it has a concentration greater than 10mg/Kg of mercury by weight. For acceptance into a Newfoundland and Labrador Landfill, mercury concentrations must be below the CCME Canadian SQGs for industrial land use (50mg/kg) or pass the Toxicity Characteristic Leaching Procedure U.S 1311 standard test for leachability for the parameters listed in Schedule II Leachate Test, CEPA proposed Regulation, 2002 (0.1 mg/L).

Recommendations:

Laboratory analysis confirmed that:

- None (0) of the four samples analyzed **contained** an arsenic concentration greater than the CCME Canadian Soil Quality guidelines for industrial land use.

- All four (4) of the samples **contained** a lead concentration greater than the provincial guideline of 0.06%.

- None (0) of the paints analyzed contained a Mercury concentration greater than 10 mg/kg.

170B Roe Avenue
Gander, NL

Bus: 709.754.4146
Fax: 709.754.4194
Email: kwall@toalltech.com

Due to the confirmed presence of lead in multiple samples, these paints must be treated and disposed of as hazardous waste, unless further lead leachate testing indicates otherwise.

If you should have any questions regarding the results and/or recommendations, please feel free to contact me at (709) 754 4146 or via email at kwall@toalltech.com

Thank You,

Kristen Wall

____
Kristen Wall, B.Tech., Env. Tech
Environmental Technician
ALL-TECH Environmental Services Limited

Reviewed by:

____
Evan Jackson, B.Sc.
Senior Environmental Consultant
ALL-TECH Environmental Services Limited

Encl: Laboratory Results (2)

EMSL Canada Inc.
2756 Slough Street, Mississauga, ON L4T 1G3
Phone/Fax: (289) 997-4602 / (289) 997-4607
http://www.EMSL.com torontolab@emsl.com

EMSL Canada Or	552215795
CustomerID:	55ATES44D
CustomerPO:	NL10455
ProjectID:	

Attn: **Kristen Wall**
All-Tech Environmental Services Limited
9 Allston Street
Unit 1
Mount Pearl, NL A1N 0A3

Phone: (709) 754-4146
Fax:
Received: 10/17/2022 10:37 AM
Collected: 10/13/2022

Project: **NL10455 Marystown Harbour Bridge, Marystown, NL**

Test Report: Lead in Paint Chips by Flame AAS (SW 846 3050B/7000B)*

Client SampleDescription	*Collected*	*Analyzed*	*Weight*	*RDL*	*Lead Concentration*
NL10455-01 *552215795-0001*	10/13/2022	10/18/2022	0.2536 g	0.040 % wt	0.91 % wt
NL10455-02 *552215795-0002*	10/13/2022	10/18/2022	0.2479 g	0.081 % wt	1.9 % wt
NL10455-03 *552215795-0003*	10/13/2022	10/18/2022	0.2431 g	0.41 % wt	9.0 % wt
NL10455-04 *552215795-0004*	10/13/2022	10/18/2022	0.2434 g	0.41 % wt	6.9 % wt

Rowena Fanto, Lead Supervisor
or other approved signatory

* Analysis following Lead in Paint by EMSL SOP/Determination of Environmental Lead by FLAA. Reporting limit is 0.008% wt based on the minimum sample weight per our SOP. "<" (less than) result signifies the analyte was not detected at or above the reporting limit. Measurement of uncertainty is available upon request. Definitions of modifications are available upon request.
Samples analyzed by EMSL Canada Inc. Mississauga, ON AIHA-LAP, LLC - ELLAP #196142

Initial report from 10/31/2022 08:54:21

EMSL Canada Inc.
2756 Slough Street, Mississauga, ON L4T 1G3
Phone/Fax: (289) 997-4602 / (289) 997-4607
http://www.EMSL.com torontolab@emsl.com

EMSL Canada Or	552215795
CustomerID:	55ATES44D
CustomerPO:	NL10455
ProjectID:	

Attn: **Kristen Wall**
All-Tech Environmental Services Limited
9 Allston Street
Unit 1
Mount Pearl, NL A1N 0A3

Phone: (709) 754-4146
Fax:
Received: 10/17/2022 10:37 AM
Collected: 10/13/2022

Project: **NL10455 Marystown Harbour Bridge, Marystown, NL**

Analytical Results

Client Sample Description NL10455-01 ***Collected:*** 10/13/2022 ***Lab ID:*** *552215795-0005*

Method	*Parameter*	*Result*	*RL*	*Units*	*Prep Date & Analyst*		*Analysis Date & Analyst*	
METALS								
Mercury by CVAA, SW-846-7471B	Mercury	68	50	µg/Kg	10/24/2022	PB	10/24/2022	PB
3050B/6010D	Arsenic	ND	2.0	mg/Kg	10/21/2022	MS	10/25/2022	MS

Client Sample Description NL10455-02 ***Collected:*** 10/13/2022 ***Lab ID:*** *552215795-0006*

Method	*Parameter*	*Result*	*RL*	*Units*	*Prep Date & Analyst*		*Analysis Date & Analyst*	
METALS								
Mercury by CVAA, SW-846-7471B	Mercury	88	50	µg/Kg	10/24/2022	PB	10/24/2022	PB
3050B/6010D	Arsenic	ND	2.0	mg/Kg	10/21/2022	MS	10/25/2022	MS

Client Sample Description NL10455-03 ***Collected:*** 10/13/2022 ***Lab ID:*** *552215795-0007*

Method	*Parameter*	*Result*	*RL*	*Units*	*Prep Date & Analyst*		*Analysis Date & Analyst*	
METALS								
Mercury by CVAA, SW-846-7471B	Mercury	ND	50	µg/Kg	10/24/2022	PB	10/24/2022	PB
3050B/6010D	Arsenic	ND	2.0	mg/Kg	10/21/2022	MS	10/25/2022	MS

Client Sample Description NL10455-04 ***Collected:*** 10/13/2022 ***Lab ID:*** *552215795-0008*

Method	*Parameter*	*Result*	*RL*	*Units*	*Prep Date & Analyst*		*Analysis Date & Analyst*	
METALS								
Mercury by CVAA, SW-846-7471B	Mercury	57	50	µg/Kg	10/24/2022	PB	10/24/2022	PB
3050B/6010D	Arsenic	ND	2.0	mg/Kg	10/21/2022	MS	10/25/2022	MS

Definitions:

MDL - method detection limit
J - Result was below the reporting limit, but at or above the MDL
ND - indicates that the analyte was not detected at the reporting limit
RL - Reporting Limit (Analytical)
D - Dilution Sample required a dilution which was used to calculate final results

APPENDIX L

Steel Coupon Testing Report

AMC - Atlantic Metallurgical Consulting Limited

TEST REPORT
DETERMINATION OF TENSILE PROPERTIES

Conducted By: Mechanical Testing Division
AMC - Atlantic Metallurgical Consulting

Client: CBCL
Halifax, NS

Date: Feb. 1, 2023

AMC Project No.: 23-AMC-020

Purchase Order No.: n/a

Item Tested: **10 Bridge Sections**

Testing Equipment:

- MTS 8500 Universal Testing Machine with ADMET MTESTQuattro Digital Electronics, Cal Due – Nov. 30, 2023
- Instron Extensometer, Cal Due – Nov. 29, 2023
- Mitutoyo Digital Calipers, Cal Due – Oct. 27, 2023

Testing Procedure: All testing was conducted in accordance with the requirements of:
CSA G40.20/G40.21: General Requirements for Rolled or Welded Structural Quality Steel / Structural Quality Steel

Results: See Results Table on next page.

Verification: THIS IS TO CERTIFY THAT THE ABOVE TESTING WAS PERFORMED ACCORDING TO REQUIREMENTS SET FORTH BY THE CLIENT AND AMC – ATLANTIC METALLURGICAL CONSULTING IN A MANNER CONSISTENT WITH STANDARD PRACTICES.

RESULTS RELATE ONLY TO THE ITEMS TESTED.

REGISTERED PROFESSIONAL ENGINEER
DATE 23/02/01
J. S. McLEOD
6710
PROVINCE OF NOVA SCOTIA

Verified By: ______________________

Jeff McLeod, P.Eng.
Manager, Mechanical Testing

DETERMINATION OF TENSILE PROPERTIES

Specimen ID.	Maximum Load (kN)	Tensile Strength (MPa)	Yield Load (kN)	Yield Strength (MPa)	Elong. (%)	Width (mm)	Thick. (mm)
#1	158.6	431	96.60	262	38	38.10	9.66
#2	292.5	400	134.6	184	55	38.16	19.17
#3	245.4	409	143.1	238	42	38.12	15.74
#4	250.7	414	113.4	187	46	38.05	15.93
#5	294.6	408	167.0	231	54	38.16	18.94
#6	157.1	433	95.20	263	40	38.08	9.52
#7	171.5	449	104.2	273	41	38.13	10.01
#8	156.6	413	101.7	268	30	38.05	9.98
#9	121.1	333	63.41	175	33	38.49	9.44
#10	120.7	380	72.78	229	30	38.07	8.34

Samples were tested in the as received surface condition.

PROJECT NAME Marystown Harbour Bridge		NUMBER 223049.10	PAGE OF
CLIENT NLDTI		SUBJECT Design Yield Strength from Samples	
DESIGNED MW	CHECKED	APPROVED	DATE

1 Determinaion of Yield Strength

→ *methodology from CHBDC A14.1.1*

Test Results:

Fy [MPa]	**Flange Sample of Rolled Member?**	**Equivalent f_y (MPa)**
262	No	262
184	No	184
238	No	238
187	No	187
231	No	231
263	No	263
273	No	273
268	No	268
175	No	175
229	No	229

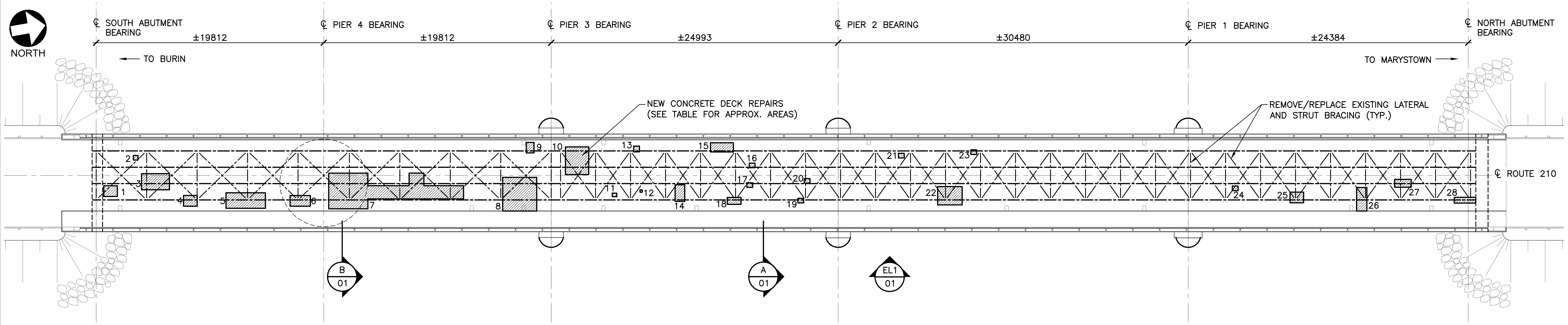
Data from Table A14.1.1	
n	**ks**
3	3.46
4	2.34
5	1.92
6	1.69
8	1.45
10	1.32
12	1.24
16	1.14
20	1.08
25	1.03
30	1

n =	10	Number of samples
f_y (avg) =	231	Average yield strength from results
Std Dev =	35.3	Standard deviation of results
ks =	1.32	
V =	0.1528	

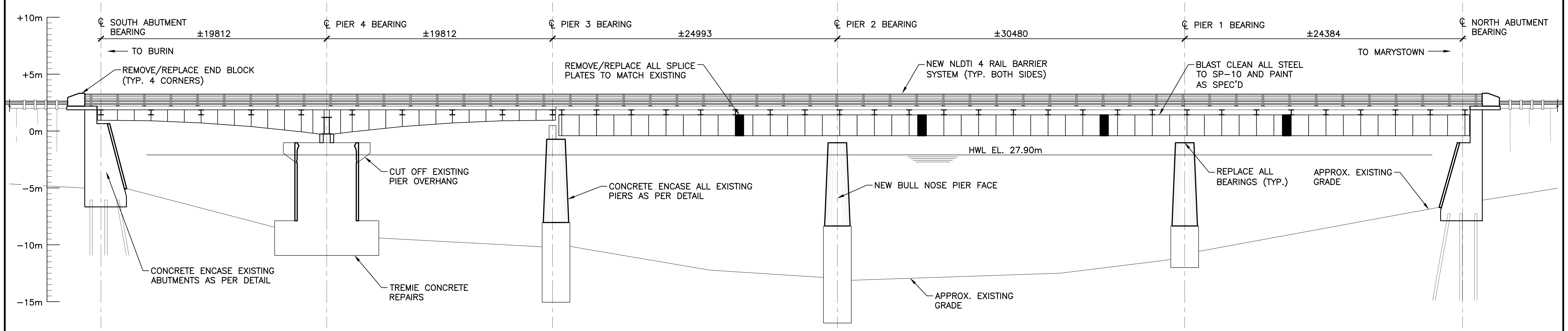
Fy = **156** MPa — Yield strength to be used in analysis

APPENDIX Q

Bridge Rehabilitation Concept Drawing



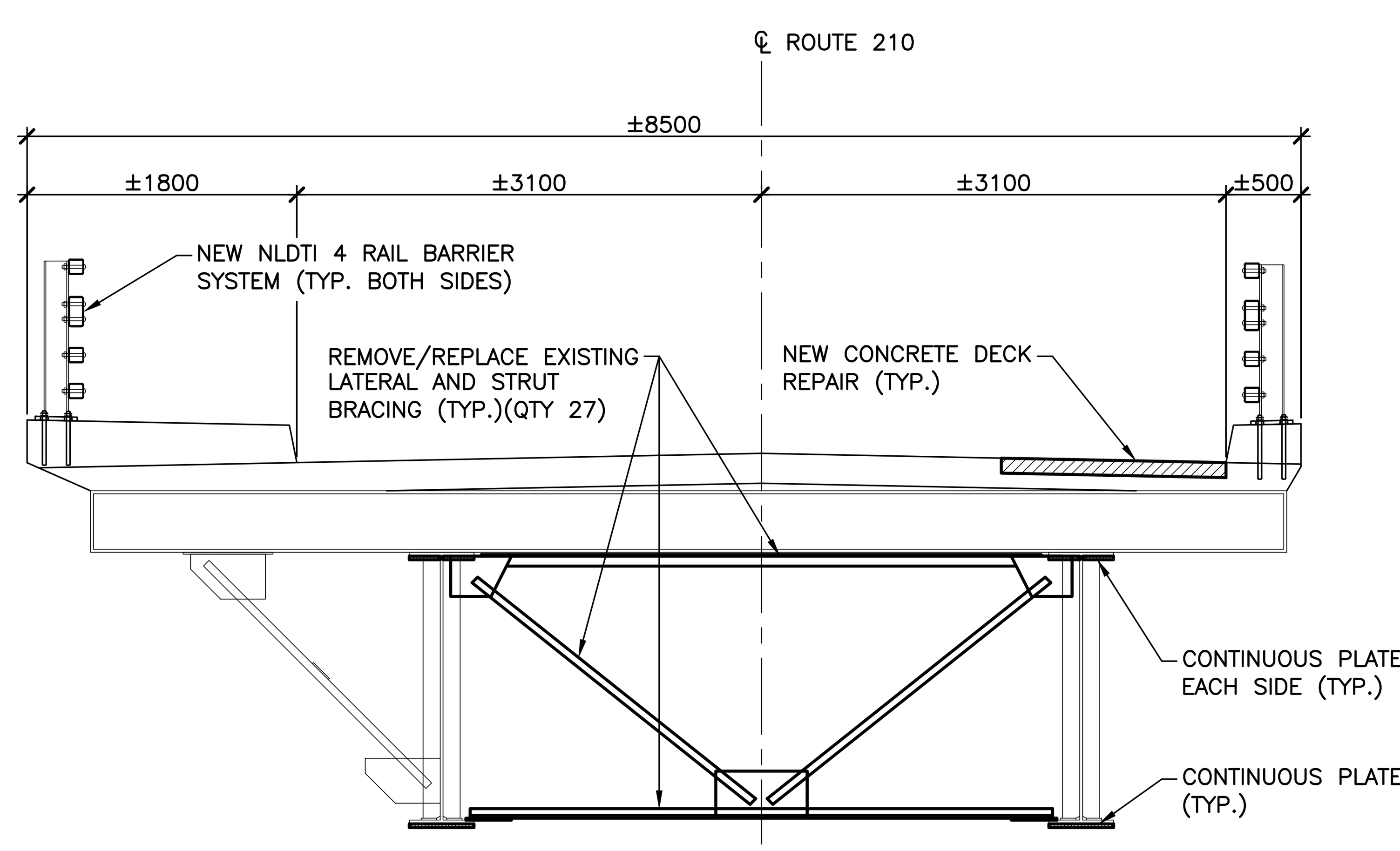
EXISTING PLAN
1:200



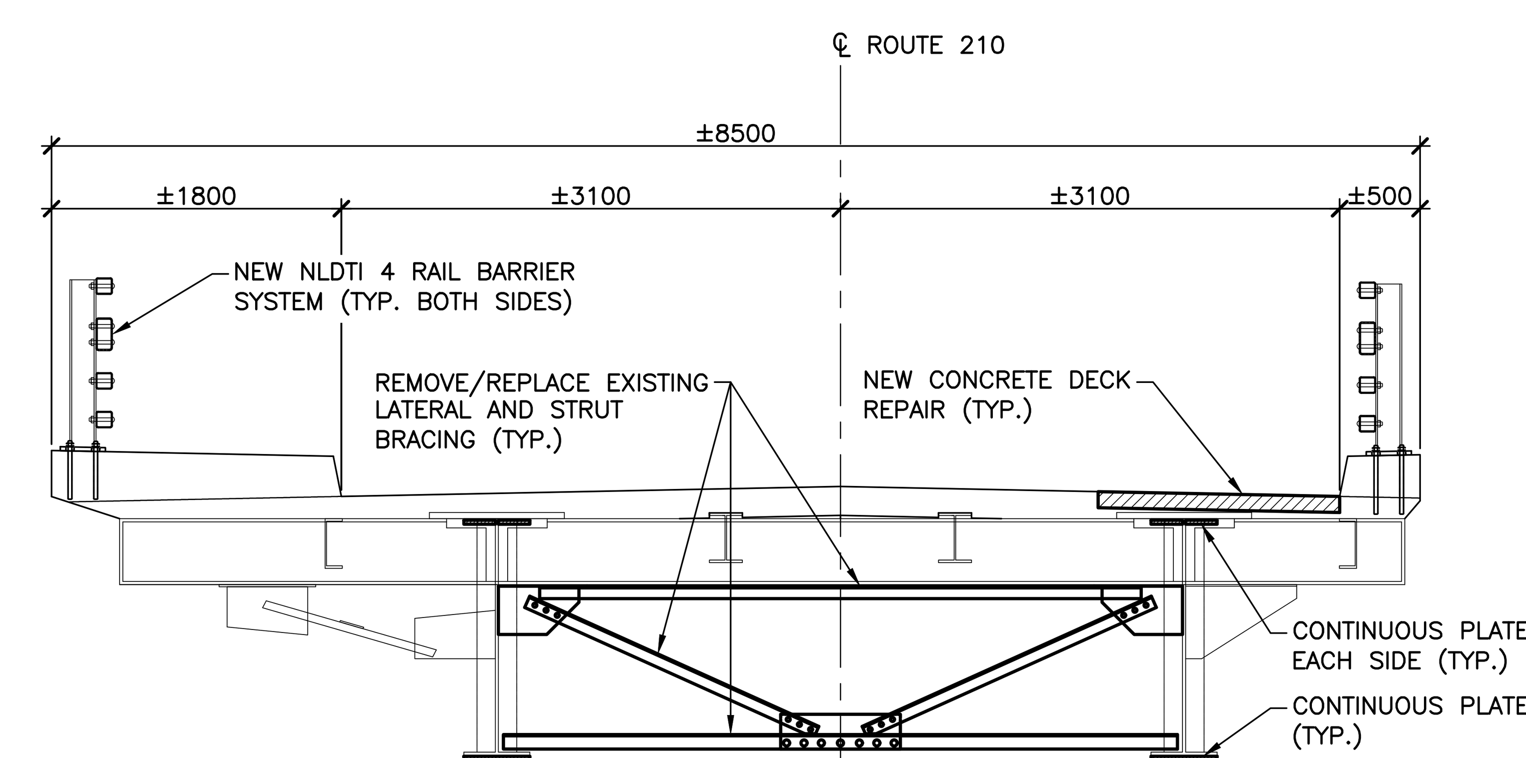
EL1 01 01 EXISTING SOUTH ELEVATION
1:200

LOCALIZED CONCRETE DECK REPAIRS			
MARK	EST. REPAIR QUANTITY m²	AS-BUILT REPAIR QUANTITY m²	AS-BUILT REINFORCING
1	0.16m²		
2	1.10m²		
3	3.40m²		
4	1.10m²		
5	4.60m²		
6	1.60m²		
7	21.60m²		
8	8.70m²		
9	0.60m²		
10	4.90m²		
11	0.15m²		
12	0.05m²		
13	0.25m²		
14	1.20m²		
15	1.60m²		
16	0.20m²		
17	0.20m²		
18	0.75m²		
19	0.20m²		
20	0.20m²		
21	0.20m²		
22	3.40m²		
23	0.20m²		
24	0.20m²		
25	1.10m²		
26	1.80m²		
27	1.00m²		
28	1.00m²		

NOTE: BASED ON PRELIMINARY ANALYSIS ONLY. TO BE REFINED IF THIS OPTION IS SELECTED.



A 01 01 CROSS SECTION – CONTINUOUS SPAN
1:40



B 01 01 CROSS SECTION – SWING SPAN
1:40

Site No. XXXX

Newfoundland & Labrador

TRANSPORTATION AND INFRASTRUCTURE
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

NOT FOR CONSTRUCTION

No.	By	REVISION DESCRIPTION	Date
A	T.P.	ISSUED FOR DRAFT REPORT	DEC. 16/22

FOR USE BY TRAFFIC ENGINEERING ONLY

Designed By	Date	Checked By	Date

DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET WHERE TAKEN
C. SHEET WHERE DETAILED

DO NOT SCALE DRAWINGS

Project/Drawing Title

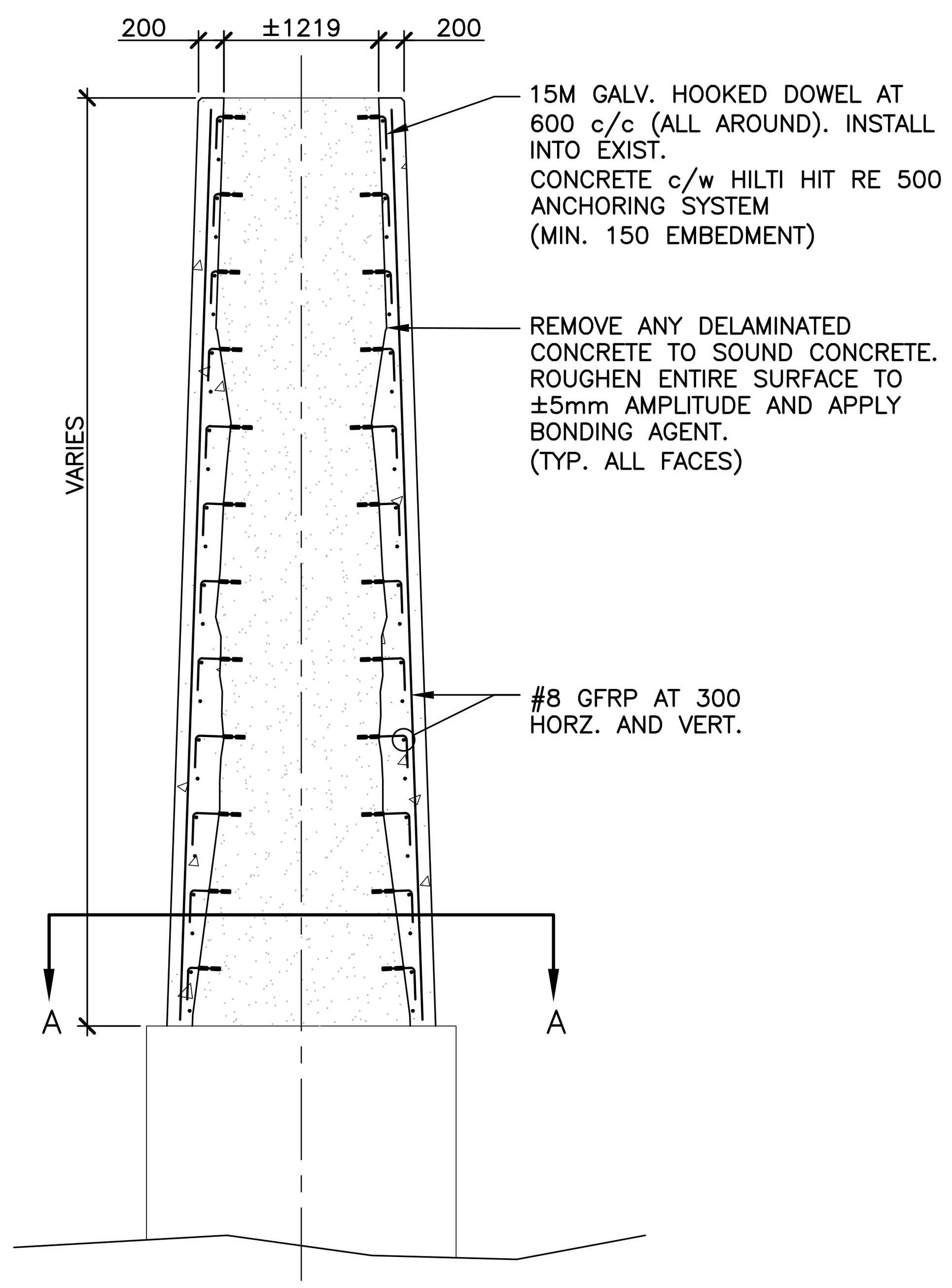
MARYSTOWN HARBOUR BRIDGE

REHABILITATION GENERAL ARRANGEMENT

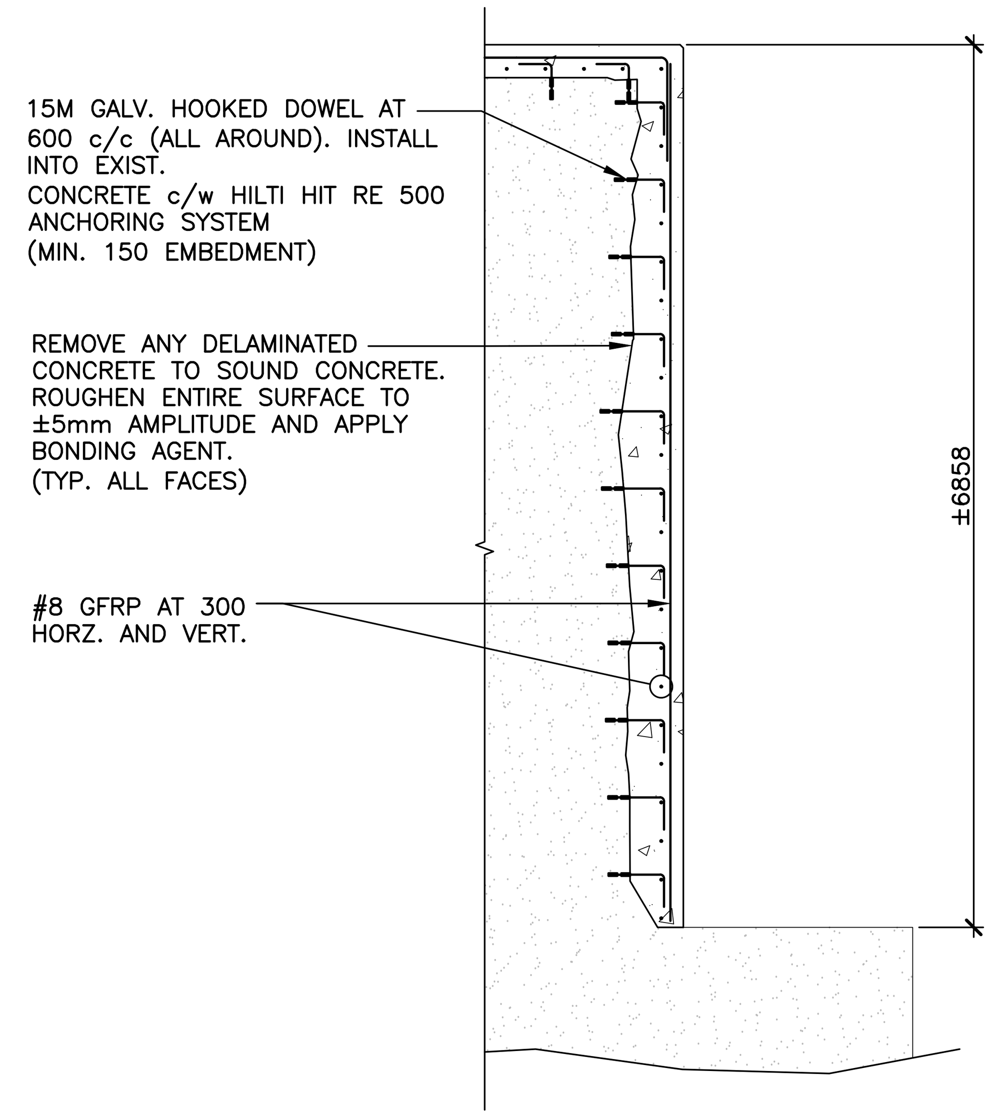
Drawn By: J. NICHOLAS
Dwg Chk'd By: M.WARREN
Designed By: T.PUDDICOMBE
Des Chk'd By: T.PUDDICOMBE
Approved By: C.JIM
Date: NOV 2022
Design Engineer

-

Sheet No. 01 of 02



SECTION PIER – CONCRETE REPAIR
1:50



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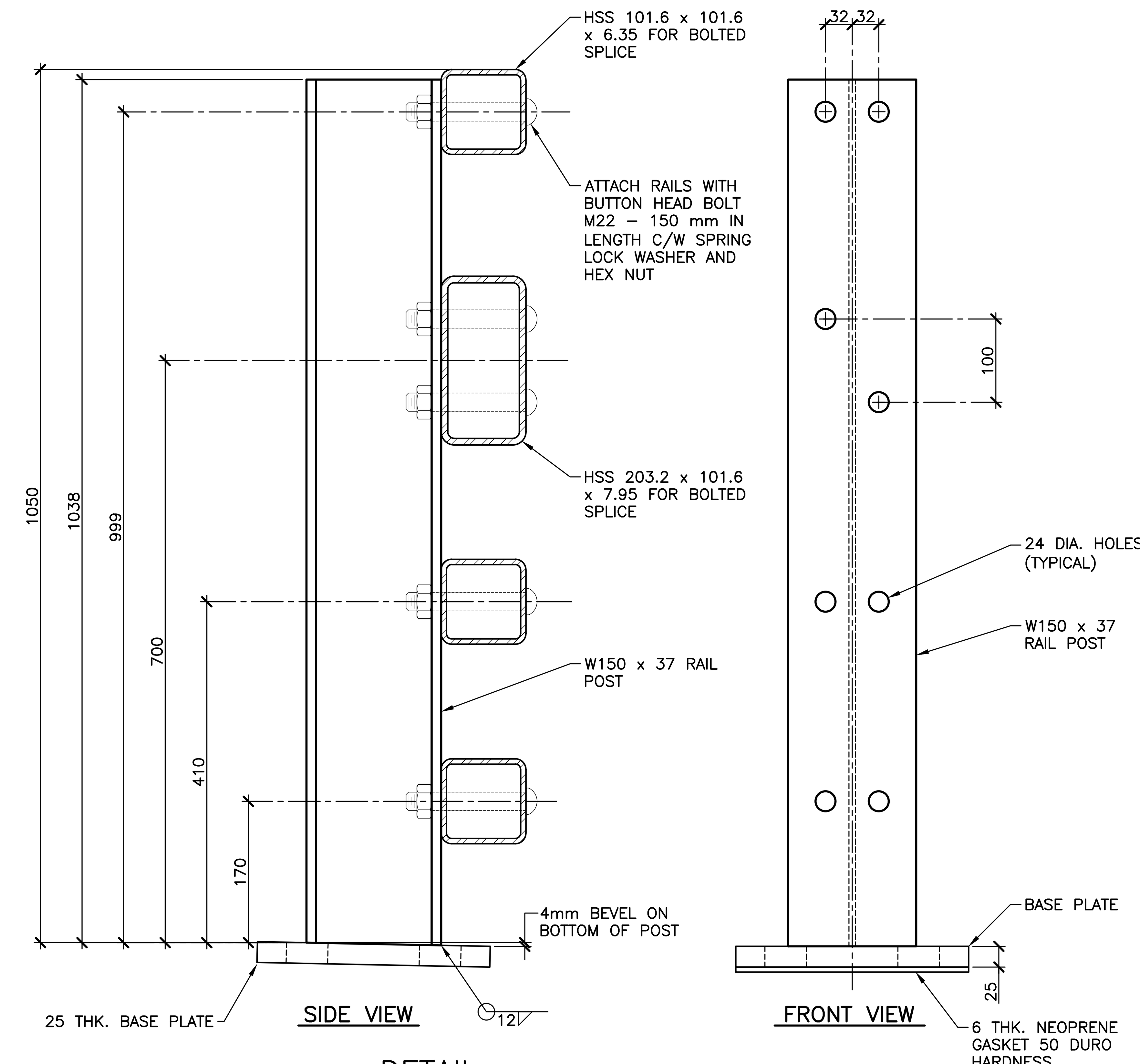
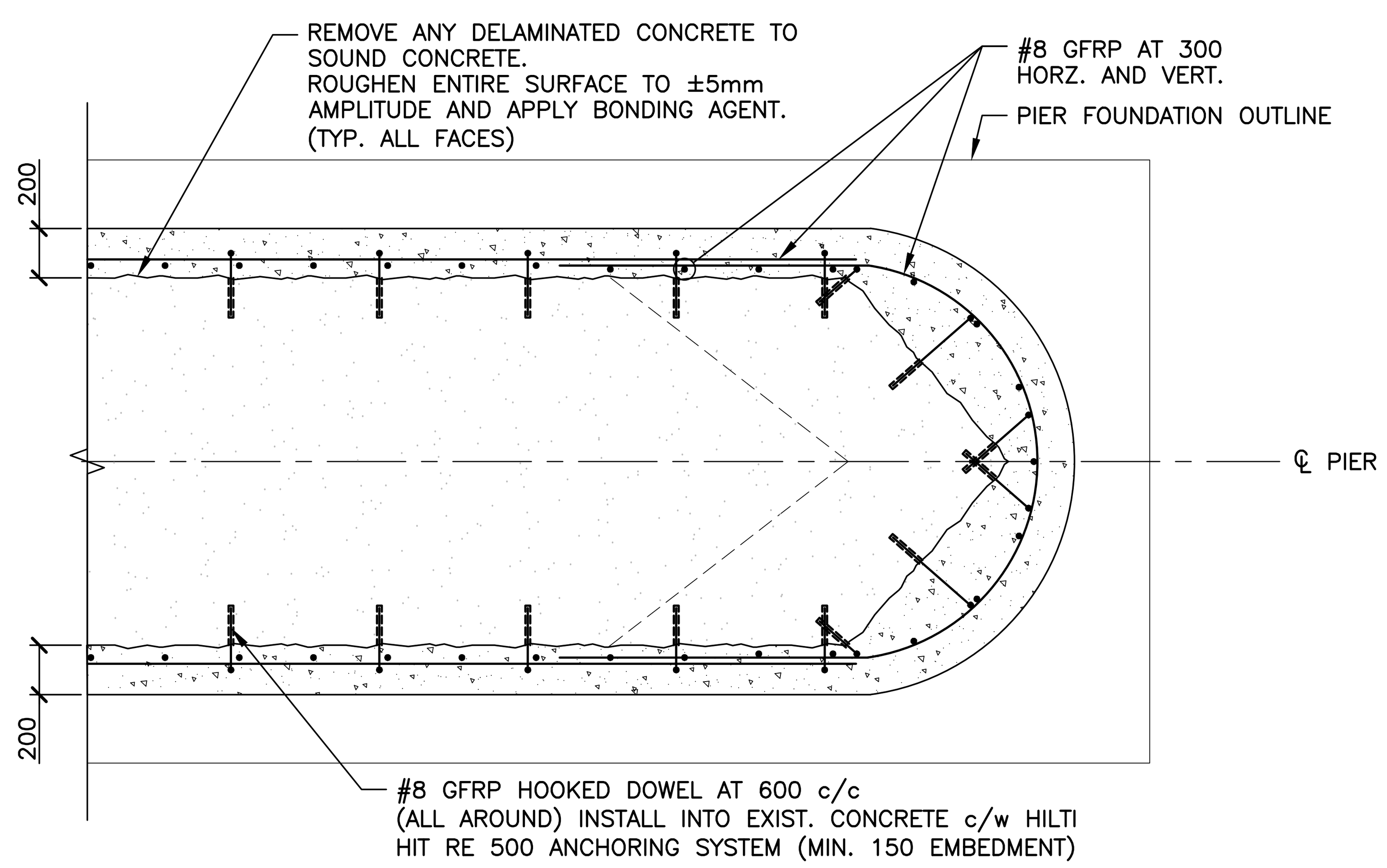


DETAIL BARRIER GUIDE RAIL POST
1:5



SECTION A PIER – CONCRETE REPAIR
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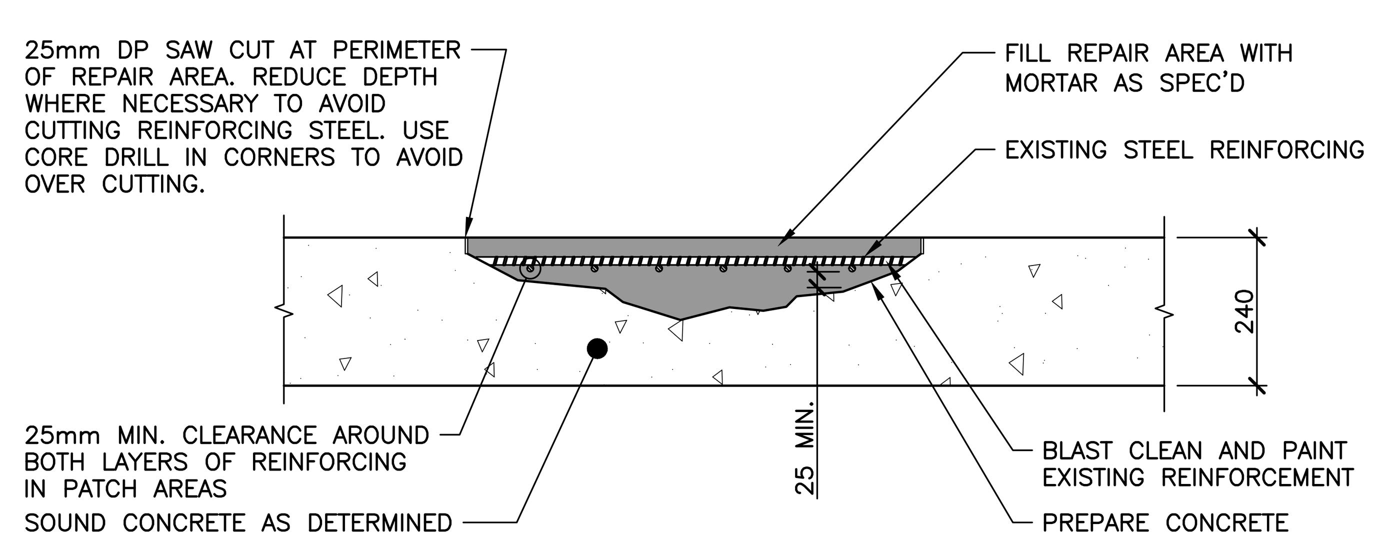
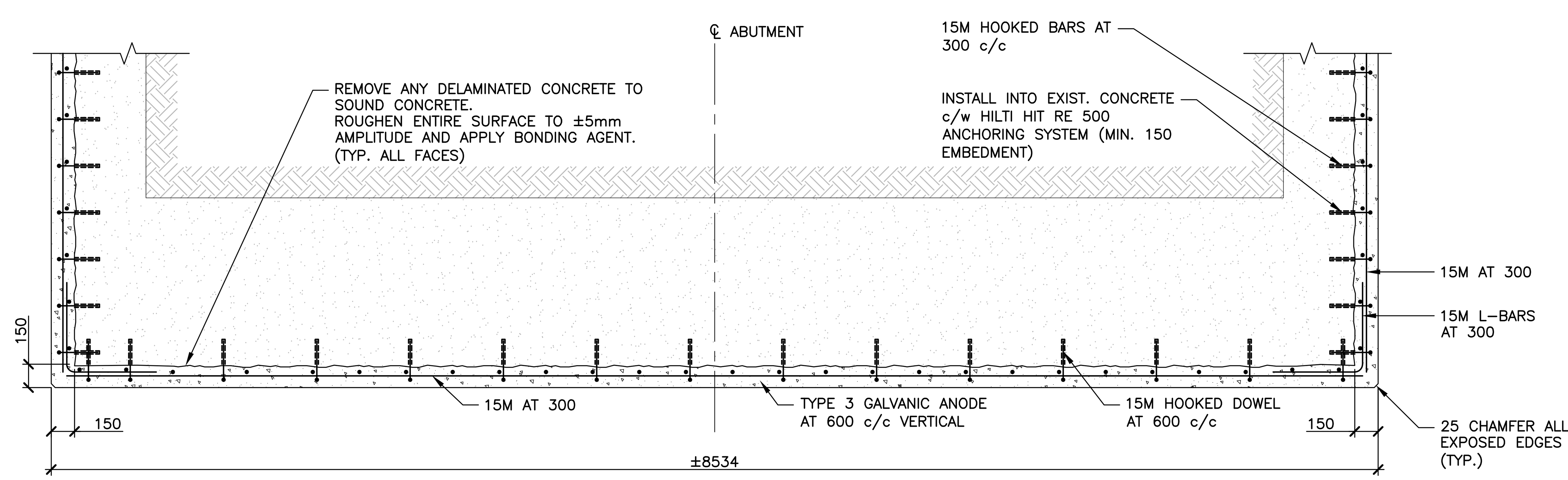


DETAIL LOCALIZED DECK REPAIR
1:10



SECTION ABUTMENT/WINGWALL – CONCRETE REPAIR
1:25

Site No. XXXX

Newfoundland & Labrador

TRANSPORTATION AND INFRASTRUCTURE
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

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A	T.P.	ISSUED FOR DRAFT REPORT	DEC. 16/22

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DETAIL SYMBOL
A. DETAIL NUMBER
B. SHEET WHERE TAKEN
C. SHEET WHERE DETAILED

DO NOT SCALE DRAWINGS

Project/Drawing Title

MARYSTOWN HARBOUR BRIDGE

REHABILITATION DETAILS

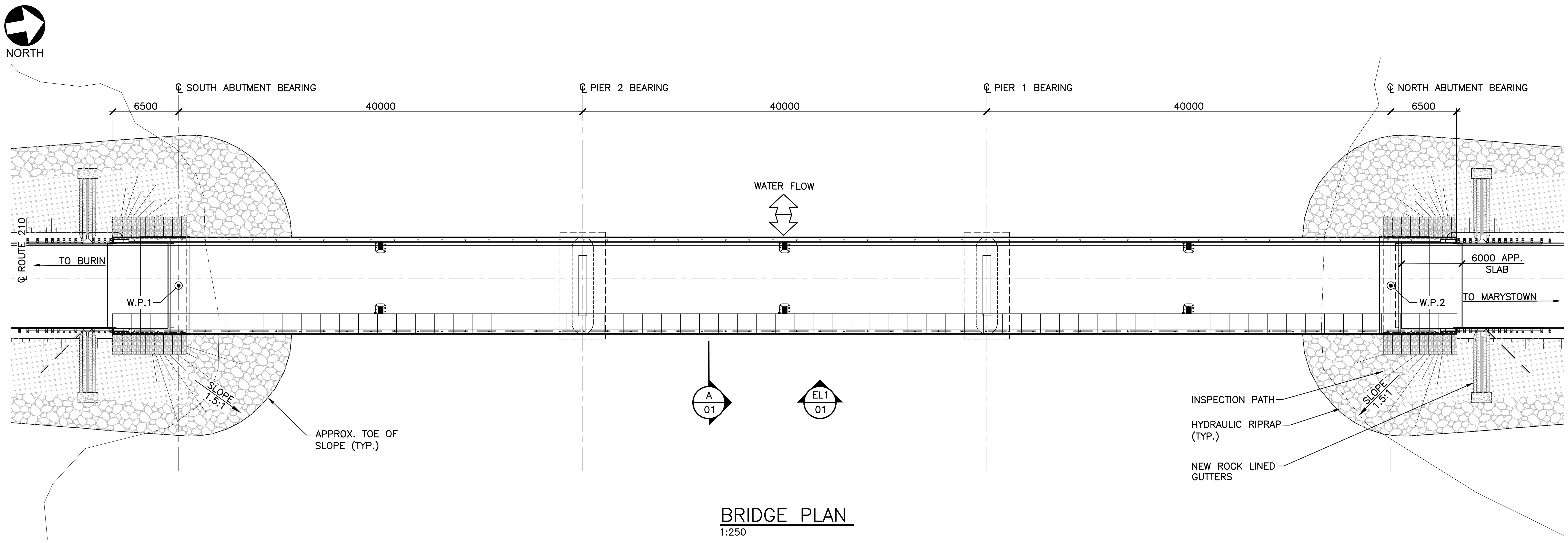
Drawn By: J. NICHOLAS
Dwg Chk'd By: M.WARREN
Designed By: T.PUDDICOMBE
Des Chk'd By: T.PUDDICOMBE
Approved By: C.JIM
Date: NOV 2022
Design Engineer

-

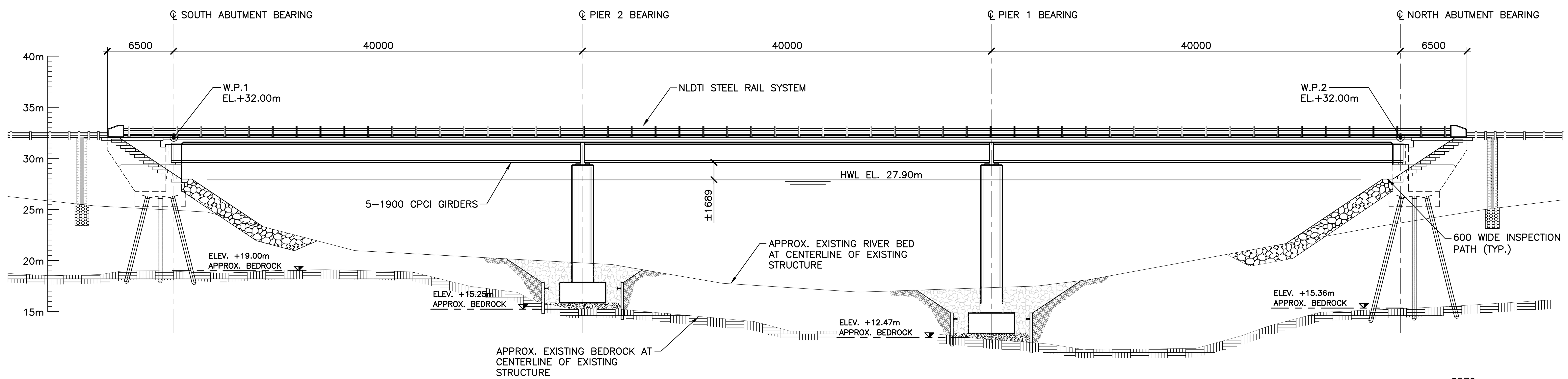
Sheet No. 02 of 02

APPENDIX R

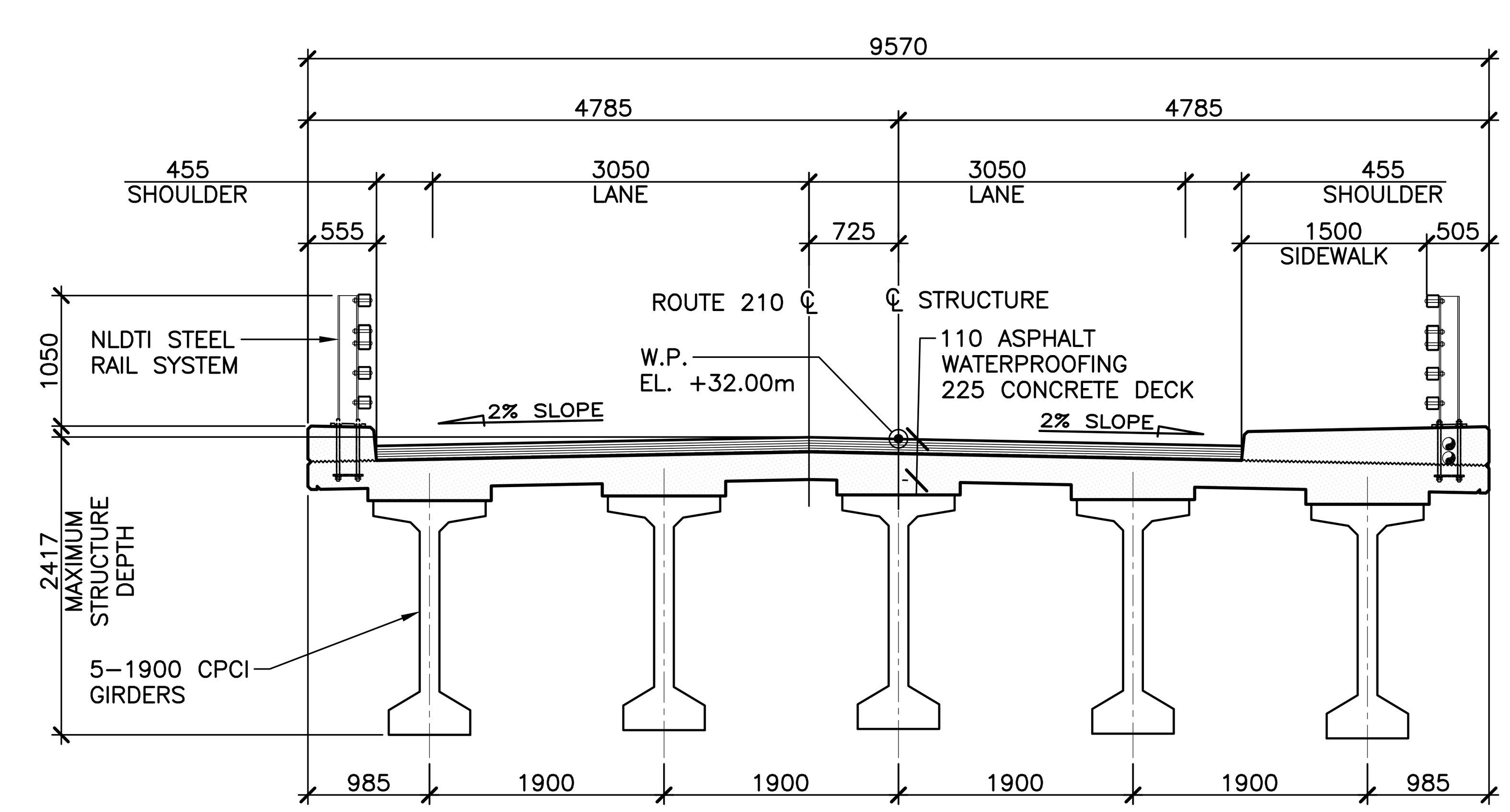
Bridge Replacement Concept Drawing



BRIDGE PLAN
1:250



EL1 01 01 BRIDGE ELEVATION
1:250



NOTE: BASED ON PRELIMINARY ANALYSIS ONLY. TO BE REFINED IF THIS OPTION IS SELECTED

A 01 01 SECTION
1:50

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Newfoundland & Labrador
TRANSPORTATION AND INFRASTRUCTURE
HIGHWAY DESIGN DIVISION
BRIDGE SECTION

GENERAL NOTES:

1. ALL ELEVATIONS SHOWN ON THIS DRAWING ARE AS PER THE ORIGINAL MARYSTOWN HARBOUR BRIDGE SUBSTRUCTURE DRAWINGS DATED APRIL 1956. ACTUAL ELEVATIONS AND EXISTING GRADE MAY VARY.

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Project/Drawing Title

MARYSTOWN HARBOUR BRIDGE

GENERAL ARRANGEMENT
REPLACEMENT CONCEPT - 1900 CPCI GIRDER BRIDGE ON SEMI-INTEGRAL ABUTMENTS

Drawn By: J. NICHOLAS	Design Engineer
Dwg Chk'd By: M.WARREN	
Designed By: T.PUDDICOME	
Des Chk'd By: T.PUDDICOME	
Approved By: C.JIM	
Date: NOV 2022	

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