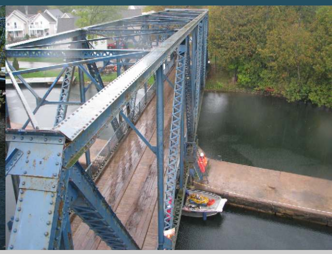
Parks Canada Parcs Canada

Hamlet Bridge (Bridge 57) Comprehensive Detailed Inspection and Structure Evaluation Report

(PCA Project No. 2011-4650-20027340)



Prepared by:

Delcan

TRANSPORTATION • INFORMATION TECHNOLOGY • WATER

March 2012

Hamlet Bridge (Bridge 57)

Comprehensive Detailed Inspection and Structure Evaluation Report

Parks Canada Project No. 2011-4650-20027340

Submitted to:

Trent-Severn Waterway Historic Site of Canada
Parks Canada Agency

Prepared by:

Delcan Corporation

In association with:

Golder Associates

March 2012

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STRUCTURAL

Swing Span

The swing span structure is in generally fair to good condition except for the structural steel coatings, the below-deck lateral bracing and the sides of the concrete pivot pier and rest piers. The steel coatings in particular should be a priority for renewal to preserve the steelwork and reduce future repair costs. Testing of the paint on the span indicated high lead levels.

Based on the structural evaluation, in its current condition the swing span could be triple load-posted to 19 tonnes for single-unit vehicles, 34 tonnes for two-unit vehicles and 43 tonnes for vehicle trains. However, the fixed span load limit is lower and so governs the load limit for the crossing, as summarized below.

The overall **Structural Condition Rating is 2 (Inadequate)**, based on the allowable load posting according to the results of the structural evaluation. The overall **Functional Rating is also 2 (Inadequate)**, for the same reason. These ratings criteria are clearly identified in the 2010 BIM.

Recommended short-term remedial work (within two years) on the span includes: Replacement of the steel cable guide rails in the west approach with MTO-approved guiderail; Maintenance of the bridge railing connections; and maintenance of approach signage.

Recommended rehabilitation work (within 5 years) on the span includes: Cleaning and re-painting of the steelwork and minor steel repairs; and replacement of deteriorated areas of timber curbs and deck.

The estimated cost of the recommended structural work is about $2.0M, including contingency and engineering costs, but excluding taxes.

Fixed Span

The fixed span structure is in generally fair condition, with the exception of the steel coatings, truss bottom chords and the east abutment. Extreme deterioration of the bottom chords was observed at the east bearings and has been addressed through installation of cables at these locations. If it is decided to maintain the current bridge in use rather than replace it, the truss bottom chords and steel coatings should be a priority for renewal to preserve the safety of the structure and reduce future repair costs. Testing of the paint on the span indicated high lead levels.

The structural review and evaluation of the span concluded that the stringers are sharing tensile load with the truss bottom chords, explaining why the span is able to support higher loads than would otherwise be possible based on the very slender and deteriorated bottom chord bars. The continuity of the stringers and their connections to the east abutment and east pier are likely a major contributor to the distress observed at the east abutment, due to restraint of thermal movements in the stringers.

In its current condition, with the bottom chords at the east end replaced or otherwise rehabilitated from their current poor condition, the fixed span could be triple load-posted to 12 tonnes for single-unit vehicles, 18 tonnes for two-unit vehicles and 19 tonnes for vehicle trains. However, if PCA decides to raise the load limit for the crossing from the current 3 tonnes, destructive testing of the connections between the stringers and east abutment should be performed to evaluate the strength and condition of the connections, prior to changing the load posting.

The overall **Structural Condition Rating is 2 (Inadequate)**, based on the allowable load posting according to the results of the structural evaluation. The overall **Functional Rating is also 2 (Inadequate)**, for the same reason. These ratings criteria are clearly identified in the 2010 BIM.

The recommended immediate work for safety reasons was the strengthening of the truss bottom chords at the east truss bearings, and this work has already been completed by PCA forces.

Recommended short-term remedial work (within two years) on the span includes: Replacement of the truss bottom chords at the east bearings; Replacement of the steel cable guiderail in the east approach; Installation of slope protection at the northeast embankment; Installation of stone rip-rap erosion protection in front of the east abutment; Maintenance of the east approach signage; and patching of approach asphalt and sealing asphalt cracks.

Recommended rehabilitation work (within 5 years) on the span includes: Cleaning and re-painting of the steelwork and minor steel repairs; Repair/replacement of the roller bearings; Replacement of deteriorated areas of timber curbs and decking; Replacement of the timber running boards on the deck; Repair of the damaged portal frame members; Concrete repairs to the east pier; Replacement of the steel connection pins at L7; and underpinning and re-facing of the east abutment.

The estimated cost of the recommended structural work is about $1.7M, including contingency and engineering costs, but excluding taxes. The estimated cost of a new bridge including a new east abutment and east pier is about $3.3M, including contingency and engineering.

MECHANICAL

All machinery is in need of cleaning and painting as a minimum. There are many fasteners with section loss and there are some failed anchor bolts which should be replaced.

With regards to the span support machinery, maintenance personnel interviewed during the inspection were unable to report the date of any internal inspection of the center pivot assembly or rehabilitation of these components. Although there were no obvious signs of problems with the center pivot assembly at the time of the inspection, consideration should be given to inspecting the wearing components of the center pivot in conjunction with any major rehabilitation work in light of the age of these components.

The balance wheel rail and anchorage is in poor condition and warrants replacement.

The arrangement of the end lift jacks does not meet the requirement of CSA S6-06, the Canadian Highway Bridge Design Code (CHBDC), that the end lift "actuating mechanism shall be non-reversible under the action of the live load." Failure of the hydraulic piping system (which has occurred previously according to maintenance personnel) would result in failure of the end jacks to support live load. This is a safety concern. The end lift jacks should be replaced with end lift machinery that meets the requirements of the CHBDC such as self-locking screw-jacks or a combination of jacks and separate end wedges.

The design of the locking pin machinery does not provide for energy absorption. There is no end of travel stop at the full open position.

Based on the behavior of the span during operation, there are no brakes or equivalent hydraulic devices (e.g. counterbalance valves) provided to hold the span stationary or allow for motion control as required by the CHBDC. A skilled operator is required to swing the span and stop it without severe impacts at the end of travel due to the limited ability to control the motion of the span. Maintenance personnel report that there have been heavy impacts in the past. Modifications to the hydraulic circuit are recommended to meet the requirements of the CHBDC and to protect the structure and machinery from impact loading.

The existing traffic gates are aged and obsolete and spare parts are no longer available.

ELECTRICAL

The bridge electrical power and control systems consist of both field located equipment and bridge control building housed equipment. The majority of the electrical control system and hydraulic system equipment was replaced during 1991/1992 upgrade and is consider as being in fair operating condition and should operate reliably with on-going maintenance in the near term (next 5-years). However, the PLC controller used for the bridge control system is obsolete and it has become difficult to obtain spare parts

The bridge is powered by the local utility from their overhead medium voltage service distribution system via a single pole mounted transformer and is in good condition. The bridge power distribution system was replaced at approximately the same time as the bridge control system and is in fair to good serviceable condition. The bridge power distribution system should operate reliably for the next 5 to 10 years with on-going maintenance.

The bridge traffic gate enclosures are in fair condition but exhibit signs of corrosion from the prevailing harsh environment. The traffic gates are of the electrically operated type and the motor controls for the gates are powered via relay contact outputs from the PLC controller. The design for the traffic gates fails to comply with the current safety standards as the gates are not operated independently using separate switches on the control console and hence cannot be directly started and stopped by the bridge operator as traffic dictates.

Operation of the bridge is via an exposed operator control station located on northeast corner of the movable span. The operator control station is provided with a cover to protect it from the harsh environment. This control station is operational but provided limited control and indication of bridge operating status. This lack of operating and indication functionality can cause a potential safety and operational hazard. Additionally, this operator control station is the only means to start and stop the hydraulic system, which causes a safety hazard for maintenance personnel when testing the hydraulic system as it can only be stopped remotely and relies on positive lines of communications between the bridge operator and maintenance personnel.

General installation of the electrical conduit, cabling and junction boxes are in fair condition with only minor signs of aging and deterioration since their installation during 1996/1997.

1. BACKGROUND

1.1 Introduction

The Hamlet Bridge (Bridge 57), owned and operated by Parks Canada Agency (PCA), carries Peninsula Point Road over the Trent-Severn Waterway in Hamlet, Ontario, north of Orillia. The crossing consists of a 31 metre fixed span and a 60 metre equal arm swing span and was constructed circa 1920. The location of the crossing is shown in the key plan in Figure 1.

In August 2011, Delcan Corporation was retained by PCA under the terms of a current Standing Offer Agreement with Public Works and Government Services (PWGSC), to complete a Comprehensive Detailed Inspection (CDI) and structural evaluation of the bridge, including structural, mechanical and electrical inspections. The mechanical and electrical inspections were undertaken for Delcan by Stafford Bandlow Engineering Inc. (SBE).

The scope of work also included non-destructive testing of selected steelwork, condition survey and materials testing of the concrete abutments and piers, a geotechnical investigation of the east abutment and underwater inspection of the piers. The non-destructive testing was performed by C.B. Non-Destructive Testing Limited (CBNDT). The geotechnical investigation and condition survey were performed by Golder Associates Limited (Golder). Underwater inspection was performed by Lower Lakes Marine.

This report documents the findings of the inspection, provides structural condition and functional ratings for the two spans of the bridge, and recommends renewal measures over the next five years and provides rehabilitation cost estimates. Completed PWGSC Bridge Inspection Manual (BIM) standard inspection forms are included in Appendix A, selected inspection photographs in Appendix B, results of non-destructive testing in Appendix C, DVDs of underwater inspection videos in Appendix D, general arrangement and defect drawings in Appendix E, paint testing results in Appendix F, geotechnical investigation report in Appendix G, detailed condition survey report in Appendix H, mechanical inspection photographs in Appendix I, electrical inspection photographs in Appendix J, structural evaluation spreadsheets and data in Appendix K, and emergency repair drawings for the fixed span in Appendix L. The mechanical and electrical inspection reports have been incorporated into the body of this report.

1.2 Description of Structure

The bridge is comprised of two spans, an east fixed span approximately 31 metres in length supported by two through-trusses (Pratt Trusses) and a west equal arm swing span about 60 metres in length also supported by two through-trusses (Warren Trusses). The bridge is a single-lane crossing with an overall width of about 5.5 metres and is currently load-posted to 3 tonnes.

According to historical articles about the site, and verified by review of the available drawings, the fixed span was originally built in 1905 for a location downstream of the existing bridge, and moved to the current location in 1915 when the existing bridge was built. Apparently construction was delayed by World War I and completed circa 1922.

The crossing has four concrete substructures: an abutment at each end, the east pier between the fixed and swing spans, the pivot pier (swing pier) supporting the swing span, and two rest piers north and south of the pivot pier. The pivot pier and rest piers are effectively one long pier.

The east pier is supported below the waterline on an original concrete core with grout-filled bags around the perimeter. The pivot pier and rest piers are supported on timber cribbing. The abutments are reportedly founded on spread footings but this could not be conclusively verified by the available drawings.

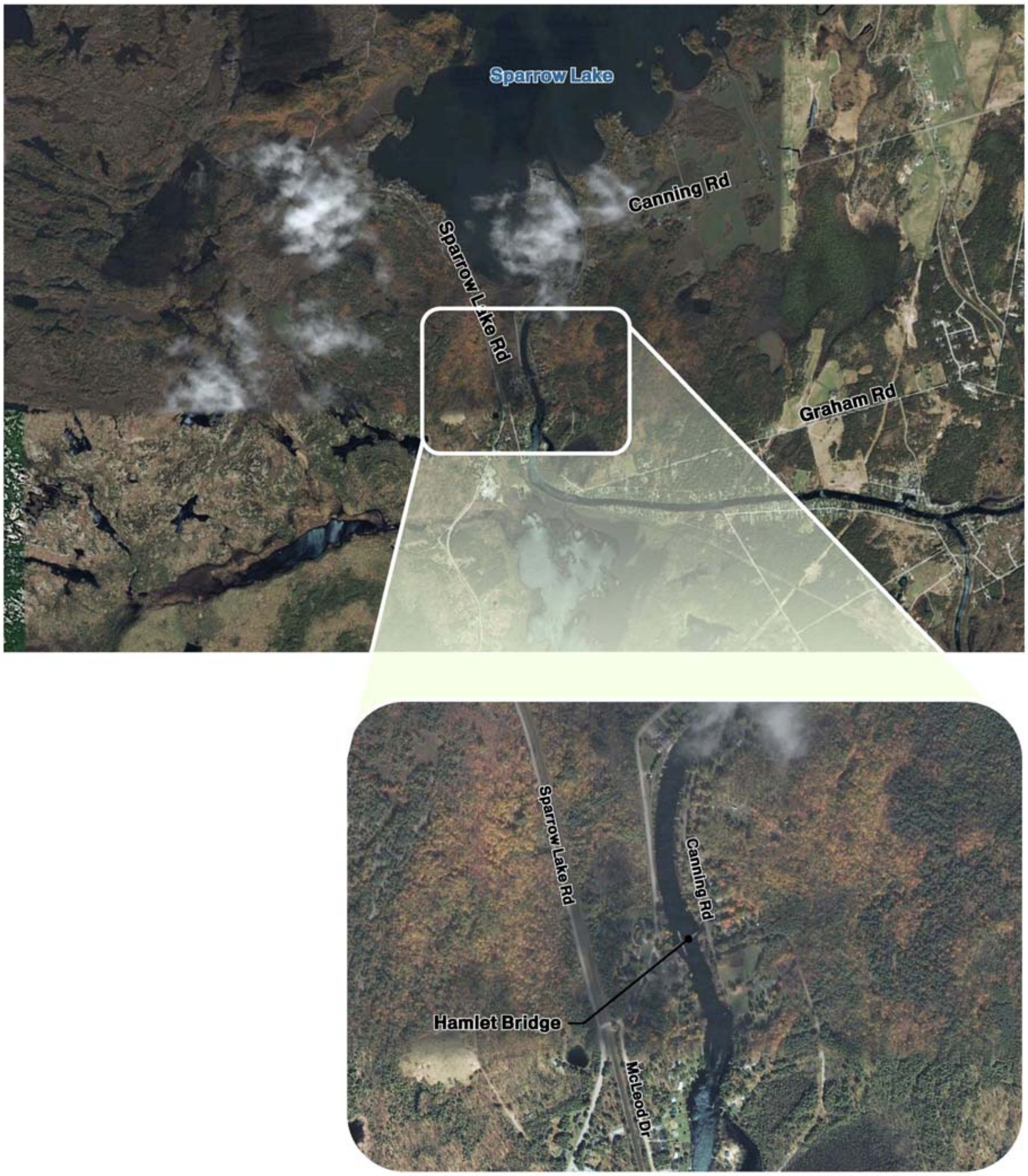

Figure 1: Key Plan

The swing span features a nail-laminated timber deck with 38 x 89 mm planks laid on edge over a steel floor system with floor beams and stringers. The truss top and bottom chords are back-to-back channels with cover plates and lattice. The web members are small I-sections and back-to-back channels with lattice. There is a central pivot bearing (pintle) that supports the entire weight of the span when it is swung open, wheel bearings at the east pier and hydraulic jack bearings at the west abutment to lift the sag out of the span ends when it is swung closed.

The fixed span features a nail-laminated timber deck with 38 x 89 mm planks laid on edge spanning transversely over a steel floor system composed of steel floor beams and stringers. The truss top chords are back-to-back channels with continuous cover plates and the bottom chords are pairs of 25 mm square bars with eye-bars at the truss panel point pin connections. The diagonal web members are square and round bars and the verticals are small I-sections. The floor beams are hung from the bottom chord panel point connection pins by 25 mm square U-bars. The stringers rest on top of, and are welded to, the floor beams. The stringers are welded to bearing plates at the east pier, and appear to be fixed to steel plate bearings at the east abutment. The span is provided with sliding bearings (roller nests) at the east pier truss bearings and fixed truss bearings at the east abutment.

Both spans have mainly riveted connections, wood plank running boards on top of the timber decking and bridge railings consisting of steel angles and lattice connected to the truss members. The fixed span also has three-pipe steel railings with steel cable below the rails.

1.3 Data Collection and Review

The following reference material was provided by PCA and reviewed during the course of the inspection and report preparation:

1. Steel superstructure, prepared by Department of Railways and Canals, entitled "Trent Canal Hamlet Swing Bridge Steel Superstructure, 200'-0" Span", dated December 1921, Dwg. No. T-2-105-4;
2. Structural steel shop drawings, prepared by Standard Steel Construction Company, dated 1922, Contract No. 1687, Dwg. No. A and B, Diagram M, Dwg No. 1 to 8, and Std-S7 to S20(except S-17);
3. Redecking plan for swing span, prepared by Indian Affairs & Northern Development National & Historic Parks Branch – Canals, entitled "Hamlet Bridge – Bridge #57, Redecking of 200' X 16'-6" Swing Span", dated June 1974, Dwg. No. TC-4417-G;
4. Control panel layout and site plan, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled "Hamlet Swing Bridge – Control Plan Layout and Site Plan", dated October 1970, Dwg. No. TC-3954-G;
5. Schematic electrical diagram, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled "Hamlet Swing Bridge – Schematic Electrical Diagram", dated October 1970, Dwg. No. TC-3955-G (superseded by TC-4149-G) and TC-4149-G;
6. Right of way plan, prepared by Trent Canal Ontario Rice Lake Division, entitled "Plan showing Canal Right of Way through Lots 6 and 7, Con II, Township of Seymour, County of Northumberland", dated February 1921, Dwg. No. A-5-308;
7. Modifications Hamlet Bridge, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled "Modifications to Hamlet Bridge Swing Mechanism, Preliminary Layout", dated November 1964, Dwg. No. TC-3209-G and TC-3210-G;

8. Modifications Hamlet Bridge, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled “Bridge #57 Hamlet Bridge, Deck and Abutment alterations to Fixed Span”, dated January 1970, Dwg. No. TC-3893-G;
9. Modifications Hamlet Bridge, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled “Method of Underpinning Abutments Hamlet Bridge”, dated September 1970, Dwg. No. TC-3940-G;
10. Modifications Hamlet Bridge, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled “Hamlet Bridge: Bracket to be used during and after Straightening Bent Vertical Member”, dated October 1970, Dwg. No. TC-3948-G;
11. Layout and substructure plan, prepared by Trent Canal Severn Division Section No.3, entitled “Layout Plan and Details of Substructure of Hamlet Highway Bridge”, dated April 1914;
12. Layout and substructure plan, prepared by Trent Canal Severn Division Section No.3, entitled “Layout Plan and Details of Substructure of Hamlet Highway Bridge (Amended Plan)”, dated July 1915, Dwg. No. C-5-286;
13. Guardrail plan, prepared by Department of Transport, Trent Canal, entitled “Plan showing in Red Location of Flex-beam Guard rails Hamlet Bridge No. 57”, , Dwg. No. TC-1713-A;
14. Repair plans, prepared by Department of Transport, Trent Canal, entitled “Plan showing Repairs to Concrete River Pier, Hamlet Highway Bridge”, dated July 1946, Dwg. No. C-5-2920;
15. Electrical layout, prepared by Department of Transport, Marine Services, Canal Division, Trent Canal System Hamlet Swing Bridge, dated September 1965, Dwg. No. TC-3331-B;
16. Plan showing control points and levels, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled “Hamlet Bridge – Control Points and Levels”, dated August 1970, Dwg. No. TC-3937-G; and
17. Plan showing mechanical swing arrangement, prepared by Department of Transport, Marine Works, Canals Division, Trent Canal System, entitled “Hamlet Bridge No.57, Mechanical Swing Arrangement”, dated October 1962, Dwg. No. TC-2872-G and TC-2873-G;

The following table summarizes significant past work carried out on the bridge since the time of construction, based on the drawings provided.

Dwg. Date	Work Indicated	Dwg. No.	File Name
July 1946	Drawing indicates approximately 850 mm thick concrete jacketing cast around the base of the pier down to riverbed level. Drawing labels original pier concrete with "Quality of Concrete is Poor". (Based on inspection, grout bags substituted for concrete below the waterline to eliminate need for cofferdams.)	C-5-2920	t2-10505.tif
October 1962	Drawings indicate the swing mechanism was modified and a motor installed.	TC-2872-G, TC-2873-G	t2-23806.tif, t2-23807.tif
November 1964	Drawings indicate modifications to swing mechanism. Added first train gear and first train pinion.	TC-3209-G, TC-3210-G	t2-23809.tif, t2-23810.tif
September 1965	Drawing indicates electrical changes to the swing motor. Time delay in swing motor was added and run push button was removed.	TC-3331-B	t2-200824.tif
January 1970	Drawing indicates the following alterations to the fixed span: Repair of roller bearings at west end; Replacement of wood stringers with steel stringers; Welding of stringers to floor beams; Addition of one "Spencer Shortspan Standard Bearing" at each stringer bearing at the east pier; Addition of new anchored concrete at the bearing seats; Replacement of timber deck with new timber deck with running strips.	TC-3893-G	t2-23811.tif
September 1970	Drawing indicates underpinning of the abutments with vertical 12" @ 65 # wide flange steel piles with concrete caps anchored to the abutments. (Not clear which if any abutments were done.)	TC-3940-G	t2-23812.tif
October 1970	Drawing depicts shop details of bracket used to straighten vertical truss members above the floor beam connections, in the fixed span.	TC-3940-G	t2-23813.tif
October 1970	Drawing indicates partial re-facing of the west abutment.	TC-3950-G	t2-23814.tif
October 1970	Electrical control panel and wiring modifications.	TC-3954-G, TC-3955-G	t40-74101, t40-74102
June 1974	Replacement of the timber deck in the swing span.	TC4417-G	t40-930.tif

Table 1 – Past Bridge Work Based on Drawings Provided

1.4 Inspection Methodology

1.4.1 General

The structural inspection, non-destructive testing, condition survey, geotechnical investigation and underwater inspection of the Hamlet Bridge were carried out on September 28 and 29, 2011 by a combined team from Delcan, Golder, CBNDT and Lower Lakes Marine. The mechanical and electrical inspections were performed on September 26, 2011 by SBE.

Inspections were performed in accordance with the 2010 PWGSC Bridge Inspection Manual (BIM), the Ontario Structure Inspection Manual (OSIM), the MTO Structural Rehabilitation Manual, the AASHTO Moveable Bridge Inspection, Evaluation and Maintenance Manual (1st Edition, 1998); the FHWA Inspection of Fracture Critical Bridge Members (Report No. FHWA-IP-86-26, September 1986); and the Occupation Health and Safety Act (OHSA). Prior to the commencement of the field inspection, a Site-Specific Health and Safety Plan was prepared and submitted to PCA. A copy of the plan was kept on site at all times with the inspection team.

1.4.1.1 Structural Inspections and Non-Destructive Testing

The structural inspection of the bridge consisted of the visual examination of exposed and accessible above-water components. Inspection was performed from accessible locations on and around the spans and substructures. A work boat was used to access the soffit of the fixed span and the sides of the piers and a boom truck was used to access the upper areas of the trusses. The soffit areas of the swing span were accessed from the rest piers with the span swung open.

The superstructures and substructures of the bridge were visually inspected to assess their condition in terms of general damage, deterioration, deficiencies and maintenance issues. Suspect areas of concrete components were sounded to detect delamination. Other structural and non-structural components, including the structural steel, coatings, timber deck, timber running boards and curbs, bearings, joints, railings, pavements, and approaches were visually inspected and their conditions were noted.

Observations and defects for the bridge components were recorded on standard BIM inspection forms and are included in Appendix A. Photographs of typical and specific defects, and the overall condition of the structure were taken for record purposes and are presented in Appendix B.

Non-destructive testing consisted of ultrasonic testing of selected steel components to determine the remaining thicknesses of sound metal, and magnetic particle testing to inspect for the presence of cracks at selected locations. Ultrasonic testing was also carried out at the connection pins in the trusses of both spans to detect the presence of any concealed cracks. Ultrasonic testing generally focused on deteriorated areas of flanges and webs in the floor systems. Magnetic particle testing was performed on selected stringer copes in both spans as most of these copes are flame-cut, square copes which sometimes develop cracks. Delcan and CBNDT representatives worked in conjunction to select locations for non-destructive testing. Refer to Appendix C for the complete CBNDT reports.

1.4.1.2 Underwater Inspection

An underwater inspection of the submerged sections of the east pier, pivot pier and rest piers was carried out by a Lower Lakes Marine diver. The inspection was carried out using a helmet-mounted underwater video camera and surface monitor to permit real-time viewing and two-way conversation between the diver and the Delcan structural engineer at the surface. All defects were noted and later recorded on drawings (see Appendix E).

1.4.1.3 Detailed Condition Survey

Golder carried out a detailed condition survey consisting primarily of materials testing of concrete cores taken from the east abutment, east pier, swing span piers, and west abutment. Refer to their report in Appendix H for additional information. The cores taken were tested for compressive strength, air void content and chloride content. Golder also assessed the piers and abutments for concrete defects such as cracking, delamination, spalling and scaling.

Six concrete core samples were taken from the east abutment and wingwalls, two cores from the northwest wingwall at the west abutment, two cores from the west abutment wall, four cores from the pivot pier (swing pier), and two cores from the east pier. The coring and reinstatements were completed by Golder personnel. Eleven cores were tested for compressive strength, four cores were tested for chloride content and two were tested for air voids.

1.4.1.4 Mechanical Inspection

SBE performed the mechanical inspection of the swing span. Mechanical systems inspected included the span support machinery, the locking pin machinery and end of travel stops, the span drive machinery and hydraulic power unit and the traffic gate machinery.

1.4.1.5 Electrical Inspection

SBE carried out the electrical inspection of the swing span. Electrical systems inspected included the electric utility service, the bridge operating electrical system, the bridge control station, the bridge end lift system, the bridge drive system control limit switches, the vehicular and marine traffic control, the cables, junction boxes and submarine cable and lighting.

1.4.1.6 Geotechnical Investigation

Golder carried out a geotechnical investigation at the east abutment consisting of three boreholes, two coreholes and one test pit. Concrete core samples were recovered from a horizontal corehole in the west face of the east abutment wall, and from a vertical corehole through the entire height of the east abutment wall. A single rock core was recovered from one of the boreholes behind the east abutment. The drilling of the boreholes, coring, and subsequent reinstatement were carried out by Golder sub-contractors under the supervision of Golder personnel. The test pit was dug south of the east abutment south wing wall by Golder personnel.

1.4.2 Personnel

The structural and civil site inspection was carried out on September 28 and 29, 2011 by Patrick Mergel, M.Eng., P.Eng., ing., Ben MacMaster, P.Eng., and Peter Harvey, E.I.T. of Delcan.

The non-destructive testing was completed by David Guest, C.E.T. of CBNDT on September 28 and 29, 2011.

The detailed condition survey was completed by S. Jagdat, P. Eng., P.Barnhill and Z.Lin of Golder on September 28, 2011. The geotechnical investigation was completed by S. Jagdat, P. Eng. and Andy Zhong of Golder on September 28 and 29, 2011.

The detailed mechanical inspection was completed by Ralph G. Giernacky, P.E. of SBE on September 26, 2011. The detailed electrical inspection was completed by Yang Feng Zheng, P. E. of SBE on September 26, 2011.

1.4.3 Component Condition Ratings

Ratings of bridge components have been undertaken in conformance with the rating system set out in the 2010 PWGSC BIM. In this system, the Material and Performance Condition Ratings (MCR and PCR) are comprised of a numerical grade assigned to each component of the structure based upon the severity of the observed material defect or the ability of a component to perform its function within the structure (refer to Appendices A and B of the BIM). The numerical rating assigned to a particular component reflects the most severe condition of material or reduction of performance observed.

Results of the inspection were summarized on the standard inspection forms located in Appendix A of this report, where the ratings of the components were determined based on the procedures of Part 2, Section 2.2 of the BIM. The Previous Condition Ratings column that was included on the 2008 BIM Comprehensive Detailed Inspection Form has now been removed from the inspection forms in the 2010 version of the BIM, and therefore these values have not been included. As no previous inspection reports were available, the Previous MCR, PCR and Priority Codes are unknown.

Each component has been assigned a Material Condition Rating (MCR) and a Performance Condition Rating (PCR), in accordance with the BIM. Tables detailing specific material and performance related defects may be found in Appendix A of the BIM. General guidelines for percentage reduction based on the severity and extent of material defects and on the reduction in capacity to perform its intended function are as follows:

Condition Rating	% Material Loss or Reduction in Capacity
6	0
5	0 – 5
4	5 – 10
3	10 –15
2	15 – 20
1	> 20

Table 2 – Condition Ratings

1.4.4 Component Repair Priority Codes

In accordance with Section 2.3 of Part 2 of the BIM, the Priority Codes assigned to each component used in the rating forms are in accordance with the following table.

Code	Description
U	Urgent; requires immediate attention and remedial measures to ensure public safety
M	Required work to be done as part of routine annual maintenance
S	Further study/investigations/surveys required prior to initiating repair program
A	Repair and/or replacement to be done in less than 1 year
B	Repair and/or replacement to be done in less than 3 years
C	Repair and/or replacement to be done in less than 5 years
D	Condition to be re-assessed at the next inspection

Table 3 – Priority Codes

2. STRUCTURAL INSPECTION

A summary of the field observations, as well as the condition ratings and repair priority codes for the individual components of the swing and fixed spans, are included on the inspection forms in Appendix A. Individual component field observations, material and performance condition ratings are included on the MCR/PCR Forms in Appendix A.

2.1 Swing Span

2.1.1 Deck Components

2.1.1.1 Deck

The bridge deck consists of 38 x 89 mm pressure-treated wood planks laid on edge spanning transversely, and connected to the floor system with galvanized steel anchors. At the east and west ends of the deck there is a larger transverse timber. The wood decking is in generally good condition with localized areas in poor condition. The timber member at the west end of the deck is rotted and split along its length and needs replacing. The central section of the timber member at the east end of the deck is rotted and split (Photo S41). The central exposed section of the deck has small holes in many areas, which appear to be areas of rot (Photo S40). There is a broken deck member between S0-3 and S0-4, midway between FB0 and FB1. The top surface of numerous deck members exhibits splitting, checking and areas of rot. The top of four members between FB4 and FB3 on the south section of deck are splitting, and the top 20 mm of six lateral members between FB0 and FB1 in the central section of deck is severely rotted. There is a 30 mm deep check in a deck member in the north area of the deck between FB8 and FB9.

2.1.1.2 Running Boards

The running boards consist of two sets of five timber boards arranged longitudinally and set approximately 1.9 metres apart centre-to-centre. The boards are approximately 50 x 250 mm and are in generally good condition, but with numerous localized areas in poor condition.

Numerous boards are severely rotted and need replacing, including the following in the north set of boards: the end 300 mm of all five boards at the west end of the bridge; #5 (south) boards at the east and west ends; board #4 at the east end; the east end of board #4 between FB0 and FB1; the west end (100 mm) of board #1 between FB1 and FB2; a 1500 mm long section of boards #3, #4, #5 at FB2. The following south boards exhibit severe rot: first and second #2 boards from the east; a 100 mm long section of board #3 between FB6 and FB7; a 1000 mm section of board #2 at FB5; board #5 between FB3 and FB4; an 1800 mm long section of board #3 at FB3; 500 mm and 700 mm long sections of board #3 at FB1; an 1800 mm long section of board #2 between FB0 and FB1. The inside 75 mm of the boards either side of the central longitudinal section of deck sound hollow and have light abrasion along the entire length of the deck (Photo S42). The plywood shim beneath the north boards at the east end of the deck is rotten (Photo S41).

2.1.1.3 Bridge Railings

The bridge railings are in generally fair condition, with numerous locations of impact damage to lattice, end balusters, and upper and lower rails noted. The coating system has typically failed on at least 30% of the lattice and 50% of the top and bottom rails, with light to medium corrosion developing.

The north railing has impact damage to two lattice on the east side of FB6, one lattice and baluster at FB5 (Photo S44), the baluster at FB4, the bottom rail and baluster at FB3, the bottom rail between FB1 and FB2 (Photo S46), and the end baluster between FB3 and FB4. Loose bolts were noted in the north railing at the bottom connection at the west side of FB8 and at the bottom connection on the east side of FB5. The intermediate railing post between FB3 and FB4 is bent (Photo S47). The south railing has impact damage to the bottom rail and lattice between FB1 and FB2, the bottom rail east of FB3, the baluster at FB5, the baluster at FB6, and two balusters at FB8. A loose bolt was noted in the south railing at the bottom connection at FB3. The bolts in the bottom rail on the west side of FB6, and in the connection to the post between FB8 and FB9, on the south railing system are missing (Photo S45).

The railing system does not meet current CHBDC crash-tested requirements or applicable provincial standards.

2.1.1.4 Curbs

The timber curbs consist of nominal 89 mm high by 140 mm wide longitudinal members, supported by 89 mm high by 140 mm wide blocks. The curbs are attached to the deck through the blocks by steel bolts.

Both the north and south curbs are in generally good condition, but light abrasion, checks and splits are typical along the length of both curb faces. The north and south members at the east end of the bridge are not tapered. The section of the north curb at the west end of the bridge is loose and is splitting longitudinally (Photo S43). The section of the south curb between FB3 and FB4 has a 6 mm wide longitudinal split along its length.

2.1.2 Superstructure Components

2.1.2.1 Trusses and Truss Connections

The truss members are generally in good condition, but the coating system is deteriorating over large areas, allowing corrosion to develop (Photo S21).

The bottom chords are in generally good condition. Areas of failed and flaking coating system were typically observed, with light corrosion developing. Member BC0S has lost approximately 40% of its coating system. Rust jacking of the top and bottom plates at the splice of member BC6S was noted.

The top chord members are in generally good condition, but with coating failure and light corrosion over 20% of the top cover plates, and 30% typically on the insides of the channels. Members TC4N and TC4S were observed to have areas of ponding on the top cover plates, and members TC5S and TC4S have moss growing on the bottom flanges.

The diagonal members are in generally good condition, but with extensive areas of coating failure and light corrosion typical. The diagonals on the north truss typically have coating failure and light corrosion over 30% of the member. D6S has light corrosion on the majority of the upper section.

The vertical members of the truss are in generally good condition, but with extensive areas of coating failure and light corrosion typical (Photos S22 and S36). Approximately 80% of the coating on the top half of member V8N has flaked off, and light corrosion has developed. V2S has coating failure and light corrosion over 50% of the inside flanges. V4S has four steel counterweights at the base, which will retain moisture and accelerate deterioration of the steel in the member.

The central bay vertical bracing members between panel points 4 and 5 are in generally good condition, with approximately 10% coating failure and light corrosion typical. The top layer of steel of the bottom diagonal bracing member in the south truss has delaminated from the angle (Photo S35).

Approximately 50% of the coating system has typically flaked off from the lower truss connections, with light to medium corrosion developing (Photo S34). L2N has light to medium corrosion over 100% of the top horizontal plate and 80% of the vertical gusset plates, and L7N exhibits light to medium corrosion over 100% of the top and bottom horizontal plates. L7S has 100% coating failure and medium corrosion on the bottom gusset plate.

The upper truss connections are in generally good condition, but typically have light corrosion over most of the top plates due to coating loss. The upper plates at connections U3N and U7s are severely bent due to rust jacking. There is a bird's nest on the lower plate of U7S.

2.1.2.2 Floor Beams

The floor beams are in generally fair condition, but there are numerous areas in poor condition, with extensive areas of coating loss and light to very severe corrosion noted on numerous members (Photo S26). Severe localized section loss and pitting of many members has occurred, including in the webs near the bottom flanges of connecting stringers, on the undersides of the top flanges, and at the bottoms of the webs at many locations (Photo S28).

FB4 exhibits very severe corrosion and knife-edging of the bottom flange and gusset plate on the west side at the connection to the bracing. FB0 has severe corrosion and deep pitting on the top flange and the web at the connection to the end stringers. Some of the areas of deterioration identified by the ultrasonic testing include: FB0 has 73% localized section loss of the east web at the connection to S0-4; FB1 has localized section losses of 31% and 49% of the west web and the east web at the connection to S0-2 respectively; FB3 has localized very severe corrosion and pitting of the web at the connecting angle to S3-2 with 56% localized section loss (Photo S30); FB8 has 100 mm long sections of 27% and 22% localized section loss of the west web and the west bottom flange respectively at the connection to S7-2.

2.1.2.3 Stringers

The stringers are in generally fair condition, but there are numerous areas in poor condition, with extensive areas of coating loss and light to severe corrosion noted on numerous members (Photos S26, S27 and S28). Areas of 20% to 30% localized section loss at the base of the web, and localized areas of medium corrosion on the underside of the bottom flange are typical.

There are typically gaps between the stringer bottom flanges and supporting shelf angles at the stringer ends. There is severe rust jacking at some of these locations (Photos S29 and S30). These brackets were likely for erection purposes and so the gaps are by design.

S0-1, S0-2 and S0-3 have severe corrosion on the top flange and medium corrosion with flaking steel in the web at the connection with FB0, and severe corrosion with flaking paint and deep pitting at the base of the web at FB1. Rust jacking of the angle supporting S0-2 at the connection to FB1 has pushed the angle down by 6 mm. S3-2 exhibits localized areas of very severe corrosion and pitting of the web and bottom flange at the connecting angle to FB3 (Photo S30). Ultrasonic testing identified the following areas of deterioration: S7-5 has a 100 mm long section of 33% localized section loss of the south web at the connection to FB8; S2-1 has a 600 mm long section of 23% localized section loss at the base of the north web.

2.1.2.4 Bracing

The bottom chord bracing members are in generally fair to poor condition with extensive areas of coating loss and light to severe corrosion typical on the majority of members. The horizontal leg of member 2N-3S has a 75 mm x 50 mm perforation and a larger area of severe section loss (Photo S33). Member 5S-6N has a localized area of severe corrosion and a 200 mm long perforation in the horizontal leg at the connection with 5N-6S (Photo S32). There is severe pitting and three small perforations in the horizontal leg of the south-east section of 6N-7S (Photo S31).

The upper sway bracing is comprised of diagonal cross bracing members, transverse bracing, and portal frames at each end of the truss. Extensive areas of coating system failure and light corrosion were observed on the majority of the members (Photo S23). The north diagonal member in the west portal frame is bent (Photo S25). Water is ponding and moss is growing in the bottom angles of many lateral bracing members. Many of the diagonal cross bracing members are bent or are sagging (Photo S24): 1N-2S is bent horizontally at 2S; 2S-3N is bent vertically at 3N; 3N-4S is bent vertically and horizontally at 3N; and member 7S-8N is slightly bent. The top connecting plate of member 5S-5N is bent due to rust jacking.

2.1.2.5 Pivot Structural Steel

The structural steel at the pivot is in generally poor condition, with extensive areas of coating failure and numerous areas of very severe corrosion and severe localized section loss noted throughout (Photos S37 and S38). Severe section loss of the bottom flange and rivet heads at the connections with the bracing members, including perforations on gusset plates, is typical (Photo S39). A localized section of the west bottom flange of the girder beneath FB4 has a 500 mm long section of severe section loss. The ends of the bracing members at the pivot are also typically severely corroded at the connections. A localized section in the north section of the bottom flange of the west hub member at the pivot has a 200 mm long section of very severe section loss. The bottom flange of the member connecting the two central hub members at the pivot has a 150 mm x 50 mm perforation. The top and bottom gusset plates connecting the central hub member and the diagonal bracing at the pivot have 50% localized section losses. The central girder was repaired and strengthened approximately 10 years ago to repair cracks seen at the bottom of web. The cracks were field-welded and vertical stiffeners were added.

2.1.3 Substructure Components

2.1.3.1 Abutment

The west abutment wall and ballast wall are in generally good condition, with no significant defects noted. A single vertical crack and rust stain were noted in the ballast wall. The wingwalls are in generally good condition. Small areas of light honeycombing and medium scaling were noted on the northwest wingwall (Photo S20).

The bearing seats at both the west abutment and east pier are in generally good condition. Accumulations of dirt and debris were noted on the east pier bearing seat.

2.1.3.2 Piers

East Pier

No significant defects or undermining of the grout bags under the east pier concrete were observed during the underwater inspection, but it was noted that the cementitious material in the bags is easily chipped by hand with a chipping hammer.

Numerous transverse cracks and areas of map cracking were noted in the above-water inclined surfaces of the concrete cap, particularly at the north and south ends, plus a large delaminated area at the base in the south-west corner. Very long and narrow horizontal areas of severe disintegration and spalling are present at the interface of the inclined and vertical surfaces of the concrete cap. Efflorescence was observed near the bottom edge of the inclined section on the west side. Several areas of severe scaling, severe disintegration and horizontal cracks with efflorescence are exhibited on the upper sections of the vertical surfaces (Photo S7).

Pivot Pier

The pivot pier (swing pier) is in generally fair condition with many localized areas in poor condition. The top surface of the pier has several very large areas of medium and severe scaling including along the west half of the balance rail, several wide cracks, and an area of cracked grout beneath the east side of the balance rail. The sides of the pier have several large areas of severe and very severe scaling (particularly at the top of the pier – see Photo S18), and very severe erosion along the length of the pier at the waterline (approximately 300 mm high on the east side and 430 mm high on the west side – see Photos S17 and S18). The concrete below the waterline on the west side of the pier is soft and was easily chipped by hand during the underwater inspection.

North Rest Pier

The north rest pier has a timber cribbing foundation, topped with 42” long by 36” high concrete blocks and a concrete cap. At the north end of the pier there is a “rest” wall and steel pipe railings. The concrete blocks generally overhang the timbers below by about 75 mm.

Numerous narrow to wide transverse cracks, large spalls, and large areas of severe scaling are typical in the top of the concrete pier cap. Spalls at the edge of the cap and a large area of severe scaling were both noted in the southeast corner. Several large areas of ponding water along the longitudinal centreline of the pier were observed: the northeast corner of the second section from the south is depressed by about 25 mm; the centre of the construction joint between the third and fourth sections from the south is sagging by 30 mm. Wide map cracks were noted over the entire surface of the rest wall at the north end.

Large areas of severe and very severe scaling and spalled concrete were observed in the sides of the concrete pier cap at numerous locations and in numerous concrete blocks (Photo S15). There are also numerous wide vertical cracks in the sides of the concrete cap, generally at construction joints or at where two concrete blocks meet, and deep spalls/disintegration at the interface of the concrete cap and the concrete blocks. Large, deep spalls in several concrete blocks have exposed the steel lifting hooks. The edges of many of the concrete blocks are rounded by erosion.

The timber cribbing under the west side of the pier is in generally good condition with no significant undermining noted. The top timber generally overhangs the timbers below by 50 mm. Only four timbers are visible at the centre of the pier. Some light to medium localized rotting was noted on the corners of several timbers. A 50 mm wide by 125 mm deep gap was noted between the ends of adjacent timbers at the riverbed, approximately 7 metres from the north end of the pier.

The timber cribbing on the east side of the pier is in generally good condition with no significant undermining noted, but several areas in poor condition were noted. On the east side the top timber generally overhangs the timbers below by 50 mm, with the timbers varying in size from 8" to 12". Several sections of rotting were observed: the end 125 mm of one of the timbers at the north end of the pier; several of the bottom timbers (one each at 5 metres, 7 metres and 12 metres from the north end) for a depth of up to 150 mm; the fourteenth timber from the top at 18 metres from the north end; 225 mm of the lowest crosstie timber at 25 metres from the north end; two crosstie timbers at 20 meters from the north end to a depth of 200 mm, and one at 23 metres to a depth of 430 mm; the ends of two crossties at 13 metres to a depth of 300 mm, and the end of the top timber at 5 metres by 175 mm. Other defects noted included: the top timber at the north end of the pier is loose; the second timber from the bottom at the north end of the pier has several 50 mm voids; the end 300 mm of the top timber at the south end is missing; the top of the top timber at 17 metres has split off; at 5 metres from the north end the 3rd timber from the top has spilt longitudinally, and the end 800 mm of another timber has split off completely.

There is a steel ladder on each side of the pier providing access for boat users. The ladder on the east side of the pier is severely deformed in the downstream direction, presumably due to ice flows (Photo S15).

The river bed to the east of the pier is covered in approximately 150 mm of silt, with up to 600 mm of silt on the west side. The river bed in the north-west corner has around 700 mm of silt cover. The water depth on the east side of the pier ranges from 6.73 m (5 m from the north end) to 4.09 m (25 m from the north end); the depth on the west side ranges from 1.30 m (20 m from the north end) to 2.13 m (5 m from the north end).

<u>South Rest Pier</u>

The south rest pier is comprised of a timber cribbing foundation, topped with 42" long by 36" high concrete blocks and a concrete cap. At the south end of the pier there is a rest wall. The concrete blocks generally overhang the timbers below by 75 mm to 200 mm. The south end of the pier has inclined steel plate armouring at the waterline.

Several narrow to wide transverse cracks (Photo S12), large areas of medium to severe scaling and several areas of ponding water along the longitudinal centreline of the pier top were noted (Photo S10). There is sagging of 25 mm at the centre of the second section from south. Severe scaling and map cracks were noted over the entire surface of the rest wall at the south end. A section of the wall has been cut out to prevent interference with the swing span when in the pen position. There are large gaps between the steel nosing plates at the south end of the pier, with a small tree growing through.

Large areas of severe and very severe scaling and spalled concrete were observed in the sides of the concrete cap at numerous locations and in numerous concrete blocks (Photo S13). There are also numerous wide vertical cracks in the sides of the concrete cap, generally at construction joints or at where two concrete blocks meet, and deep spalls/disintegration at the interface of the concrete cap and the concrete blocks (Photo S16). Large, deep spalls in several concrete blocks have exposed the steel lifting hooks

(Photo S13). The edges of many of the concrete blocks are rounded by erosion. There is a large void beneath the steel nosing plates at the south end (Photo S14).

On the west side of the pier only three timbers are visible at the centre of the pier. The top timber generally overhangs the timbers below by 50 mm. The timber cribbing on the west side is in generally good condition with no undermining noted, but the ends of the crossties at the south end are typically rotten.

The timber cribbing on the east side of the pier is in generally good condition, but several areas in poor condition were noted. There is a 330 mm high section of undermining at the south corner of the east side of the pier, which tapers to zero over a length of approximately 2.5 metres to the north. There are several 250 mm x 250 mm voids (one each at 13 metres, 15 metres and 25 metres from the south end) in the cribbing where the ends of longitudinal timbers have rotted away. The top timber generally overhangs the timbers below by 50 mm.

The river bed to the east of the pier is covered in large rocks and sections of concrete. Water depth on the east side ranges from 4.17 m (south end) to 2.08 m (north end); the depth on the west side ranges from 2.44 m (5 metres from the south end) to 1.93 m (15 metres from the south end).

Refer to the deterioration drawings in Appendix E for locations of defects in the pier caps, blocks and timber cribbing.

2.1.4 Structural Steel Coating System

The structural steel coating system is in very poor condition throughout the structure, with extensive areas of cracked and flaking coatings typically noted, permitting light to very severe corrosion to develop on the trusses, bracing, floor system, and pivot steel members. Apparent red lead primer was noted on many surfaces.

2.1.5 Miscellaneous Components

2.1.5.1 Expansion Joints

The open expansion joints at the east and west ends of the structure allow moisture, dirt and debris to accumulate on the bearing seats and below-deck structural steel members.

2.1.5.2 Approaches

The west approach asphalt wearing surface is in generally good condition, but does exhibit areas of light ravelling along the centreline and on the south side. Light abrasion was noted on the top of the ballast wall. The top of the ballast wall is sloped to allow smooth passage onto the bridge from the approach, but creates an uneven ride for vehicles.

The steel cable and timber post guide rails on the north and south sides of the east approach are in poor condition. On both the northwest and southeast guide rails the cables are sagging (Photo S48). The west end of the steel cable on the south side of the approach is attached to a road sign post (Photo S50). The first ten posts at the east end on the south side are severely rotted (Photo S49).

The guiderails do not meet current provincial standards.

The steel tube railings on the north side of the west approach have slight impact damage and small areas of coating failure. The steel tube railing posts on the south side of the approach typically only have two of four anchor bolts installed.

2.1.5.3 Embankments and Slope Protection

The southwest embankment is in generally good condition with no significant erosion noted. Approximately 10% of the northwest embankment at the end of, and adjacent to, the northwest wingwall has been eroded due to water runoff from the roadway. There is a tree growing near the wingwall. There are also small trees growing in front of the west abutment wall. Some minor erosion of the embankment material in front of the west abutment wall was also observed (Photo S19).

The slope protection at the west embankment is provided by intermittent rock protection. Some of the rocks appear to have been displaced.

2.1.5.4 Utilities

The old navigation light at the south-west corner of the truss is broken.

2.1.5.5 Signs

The street name and traffic light sign posts on the west approach are not vertical, possibly due to impact damage (Photo S5). The bottom bolt is missing from the “slippery road” sign at the west end of the north truss (Photo S51). The “hazard close to edge of road” sign at the west end of the south truss is loose and has some impact damage. The “stop here on red signal” sign on the west approach is loose.

2.2 Fixed Bridge

2.2.1 Deck Components

2.2.1.1 Deck

The wood deck is in generally good condition, with localized areas in poor condition. The transverse beam at the west end of the deck is rotted (Photo F24). The central exposed section of the deck has small holes along the entire length, which appear to be areas of rot. Light splitting and areas of light rot were observed at numerous locations, with some light end splitting also noted. There are accumulations of dirt and debris on the deck, predominantly on the north section.

2.2.1.2 Running Boards

The running boards on the deck are in generally good condition, with localized areas in poor condition. Minor splits, checking and wear are typical along the length of the north and south sections. The boards at the east and west ends are generally rotted and need replacing. Numerous other intermediate boards along the length of the deck have long sections of severe rotting (Photo F26). The inside edges (approximately 75 mm wide) of the boards either side of the central longitudinal section of the deck sound hollow and typically have light abrasion along the entire length of the deck.

2.2.1.3 Bridge Railings

Only the westernmost panel of the original railing system with decorative lattice remains; the remainder of both the north and south railing systems has been replaced with a three-rail steel pipe railing system with a steel cable at the base.

Neither railing system meets current CHBDC crash-tested requirements or current provincial standards.

The two remaining sections of the original railing system are in fair condition. The coating has failed on at least 50% of the railing system, with extensive areas of light corrosion (Photo F33). The south panel is bent around the end diagonal on the south truss. The hook connecting the bottom rail to truss member D6-S has severe impact damage, and the angles connecting the top and bottom rails to truss member V6-N are bent.

The steel pipe and steel cable railing system is in generally good condition. However, the angles connecting the south cable to the post at FB2 and FB5, and the north rails to the post at FB3 and FB6 are bent. Minor impact damage to the top rail was noted at several locations. Approximately 2% of the coating system has failed, which has enabled light corrosion to develop.

2.2.1.4 Curbs

The timber curbs consist of nominal 89 mm high by 140 mm wide longitudinal members, supported by 67 mm high by 140 mm wide blocks. The curbs are attached to the deck through the blocks by steel bolts. Both the north and south curbs are in generally good condition, but minor abrasion, checks and splits are typical on both curb faces.

In the north curb, the section to the west of FB2 has a 25 mm wide end split, and the end 400 mm of the member at the east end is severely rotten.

On the south curb, the member at the east end has almost entirely rotted away around the anchor bolt (Photo F27). A 25 mm wide split in the curb member to the west of FB3 and impact damage to the steel connecting bolt was observed. The spacer block beneath the south curb at the west end of the bridge has split into two separate pieces (Photo F28).

2.2.2 Superstructure Components

2.2.2.1 Trusses and Truss Connections

The condition of the truss members ranges from poor to good.

The bottom chords, which are pairs of 25 mm square bars, are in generally good condition except at the I-bar end connections, where they are in generally poor localized condition, with medium to severe corrosion typical.

The bottom bars of the I-bars at the east bearings of the north and south trusses are exhibiting extreme section loss (greater than 90%), with only approximately 1/16th of the original cross sections remaining (Photos F19 and F20). These locations were buried in debris and exposed to moisture and organic material prior to our inspection. Delcan inspectors cleaned off the bearings and identified the extreme bottom chord section losses at these locations.

Severe corrosion of the bottom chord I-bars at the west end of both the north and south trusses has also occurred, with 30% and 40% localized section losses respectively (Photo F21). There are 30-40% section losses typical in the bottom I-bars at most of the truss panel points. The bottom chord members away from the connections typically have light corrosion over 10% of the member, caused by localized coating loss.

The top chord members are in generally good condition. Approximately 10% of the coating system has typically flaked off from the top cover plate and channels, permitting light corrosion to develop (Photo F3). Severe rust jacking of the top chord cover plate at U6S has occurred (Photo F12).

The intermediate chord members, which are round bars, are in generally good condition. Approximately 5% of the coating system has typically flaked off from the members, permitting light corrosion to develop.

The diagonal members are in generally good condition. Diagonal members L2N-U3N and U3N-L4N have turnbuckle splices near the bottom of the member, and there is a clamped splice at the base of U4N-L5N (Photo F18). There is impact damage to the inside I-bar member of U2N-L3N (Photo F17). Approximately 5% of the coatings have typically flaked off permitting light corrosion to develop.

The vertical members of the truss are in generally good condition. However, at the base at the connections to the bottom panel point pins they are in locally poor condition, with severe localized corrosion and section loss of the inside flanges typical. Ultrasonic testing indicated that V4N has 70% localized section loss of the south-east flange, and V5N has 35% section loss of the south-east flange at this location. Localized areas of 10% (south truss) to 30% section loss (north truss) of the interior flange at the connection with the diagonal member of the lateral bracing are typical (Photo F14).

Member V1N is twisted about its longitudinal axis along its full length (Photo F16), and member V6N is bent at the base, possibly due to previous impact damage (Photo F15). The coating system is typically cracking and peeling at the base of the vertical members.

Approximately 50% of the coating system has typically flaked off from the lower truss connections, with light to medium corrosion developing (Photo F22). The upper truss connections typically have light corrosion over 20% of the top plate, and 5% overall. The top plate at U6S is severely bent due to rust jacking.

2.2.2.2 Floor Beams

The floor beams are generally in fair condition. Extensive areas of coating failure and light to very severe corrosion were observed on the majority of the floor beams (Photo F23). Many of the floor beams also exhibit severe localized section loss, typically on the top flanges at the connections to the stringers, and on the webs and bottom flanges at many locations.

Ultrasonic testing of the floor beams indicated severe localized deterioration of several floor beams, including: FB2 has 50% localized section loss of the east top flange at the connection to S7; FB3 has 58% localized section loss of the west face of the web at the connections to S3 and S4; and FB4 has up to 45% section loss of the east face of the web at the connections to the stringers.

2.2.2.3 Stringers

The stringers are in generally fair condition. Approximately 20% of the coating system has typically failed on each stringer, permitting light to medium corrosion to develop (Photo F23). Some inter-coat delamination has also occurred, exposing the red primer coat at many locations.

The stringers are continuous over the floor beams with welded splices at the end connections. They are also welded to the floor beams. The stringers, when viewed from the east pier below deck, were observed to not be in completely straight lines. It is not clear whether they were installed this way, or whether this has happened subsequently (Photo F25).

2.2.2.4 Bracing

The below-deck lateral bracing, consisting of round bars, is in generally good condition. However, at least 50% of the coating system has typically failed, and light to medium corrosion with light pitting has developed. There are also large areas of inter-coat delamination which has exposed the primer coat.

The upper sway bracing is comprised of round bars, built-up transverse struts, and portal frames each end of the truss. The diagonal bracing members are in generally good condition, but there are large areas of coating failure and light corrosion on all members. The portal frames are in fair condition. Impact damage has shifted the bottom lateral member of the west portal frame up and to the east by around 150 mm, and the top lateral member is bent at the south end. The bottom flange of the top lateral member of the east portal frame is bent at the south end. Up to 30% of coating has typically flaked off leading to light corrosion. The transverse bracing members are typically in fair condition. The west horizontal flange of member 4S-4N is deformed along the entire length (Photo F13). Rust jacking at the connection of inclined lateral bracing member 5S-5N and V5N has bent the connecting plate. Top lateral member 5S-5N has 10% localized section loss at the interface with the south top chord.

2.2.2.5 Pins and Hanger Bearings

The bolts and pins at the truss connections are in generally good condition. Ultrasonic testing of these elements did not reveal any cracks. However, the pins and pin casing at the east ends of the bottom chords exhibit very severe corrosion and some section loss (Photo F19 and F20).

2.2.3 Substructure Components

2.2.3.1 Abutment

The east abutment is in generally fair condition. Large areas of severe scaling and spalling are typical at either end of the abutment wall below the horizontal construction joint (Photo F7). Smaller areas of spalling and cracks were noted throughout.

Based on the findings from the Golder geotechnical investigation, the top of the abutment wall has tilted west towards the river, and the top of the south-east wingwall has tilted towards the south (see Golder geotechnical report in Appendix G). According to Parks Canada staff, the gap between the back of the east abutment ballast wall and the approach asphalt has opened up in the past (Photo F8).

The majority of the east bearing seat below the deck is covered in dirt and debris. Several wide vertical cracks and areas of spalling and delaminated concrete were observed in the ballast wall (Photo F9).

The wingwalls are in generally fair condition. The northeast wingwall has wide gaps at the horizontal and vertical construction joints, with some vegetation growing through. Localized areas of spalling and disintegration, and some wide cracks were also noted (Photo F5). The south-east wingwall has wide gaps at the horizontal construction joints, an area of spalling/disintegration at the base of vertical construction joint, and several medium cracks (Photo F6).

2.2.3.2 Piers

See Section 2.1.3.2.

2.2.4 Structural Steel Coating System

The structural steel coatings are in localized poor condition over most areas of the fixed span. Extensive areas of inter-coat delamination, and cracked and flaking coatings were typically noted on the floor system members, bracing, bottom chord connections, and on the truss members (Photo F3).

The top coat on a section of the south truss is a darker shade of blue than the top coat on the north truss. Apparent red lead primer was also observed on the span.

2.2.5 Miscellaneous Components

2.2.5.1 Expansion Joints

The open expansion joints at the east and west ends of the structure allow moisture, dirt and debris to accumulate on the bearing seats and below-deck structural steel members.

2.2.5.2 Bearings

The north and south roller bearing assemblies at the west end of the bridge exhibit light to medium corrosion with pitting and accumulations of dirt and debris. The west roller on each bearing has moved to the west of the top and bottom bearing plates and is twisted about its axis, suggesting that the roller assemblies have become detached (Photos F29 and F30).

The fixed bearings at the east end of the bridge exhibit light to medium corrosion with pitting, and are covered with accumulations of dirt and debris.

2.2.5.3 Approaches and Guiderails

The asphalt pavement in the east approach is in generally fair to good condition, but there are unsealed wide transverse cracks at the east end of the approach and several patch repairs. Several areas of ponding water were observed, indicating depressed areas. Bridge maintenance staff indicated that gaps have opened up in the past between the west end of the approach and the east ballast wall, which may be due to the possible movements of the east abutment or superstructure. These gaps have been patched with asphalt (Photo F8).

The steel cable and timber posts guide rails in the east approach are in poor condition. On both the northeast and southeast guide rails there is a significant loss in the steel cable tension, and the two end posts are severely rotted (Photo F34).

The guiderails do not meet current provincial standards.

2.2.5.4 Embankments and Slope Protection

The northeast embankment has a localized area of severe erosion at the end of the north-east wingwall, caused by water draining from the east approach, which has eroded the embankment material around the timber post at the west end of steel cable guide rail (Photo F32). The other embankments are in generally good condition, with no significant erosion or other defects noted.

The erosion protection at the east embankment is provided by intermittent rocks. The majority of the erosion protection rocks in front of the east abutment sheet piling appear to have been washed away (Photo F7).

2.2.5.5 Signs

The "Slippery Road" sign on the southeast corner of the truss has impact damage and is also loose. There is also impact damage to the "Hazard Close to the Edge of Road" sign at the southeast corner of the truss.

2.3 Emergency Repairs

As noted above in Section 2.2.2.1, very severe section losses were identified by Delcan inspectors in the fixed span bottom chord eye-bars at the east abutment bearings. Normally in trusses such as those in the fixed span, the bottom chords are "fracture critical" members, meaning that if they fail, the span will collapse. Accordingly, it was decided that the bridge should be closed until emergency repairs could be carried out. PCA acted to close the bridge the same day.

PCA subsequently retained Delcan to design emergency repair measures for the span. A system of steel cables was proposed to augment the strength of the truss bottom chords in the most easterly truss panels. The drawings for the repairs are included in Appendix L. The repairs were carried out by PCA forces and the bridge was re-opened about two weeks after closure.

3. GEOTECHNICAL INVESTIGATION

The Golder geotechnical report for the investigation carried out at the east abutment is included in Appendix G. A summary of the report is as follows:

Apparent Movements

Some discussion is presented about the gaps between the fixed span and swing span railings at the east pier, and it is stated that the bridge operator reported that the gaps have decreased in size over the past several years. It is also stated that the bridge operator reported that gaps between the back of the east abutment ballast wall and approach asphalt have developed in recent years. As noted previously, there are currently asphalt patches at this location.

Plumb lines were dropped from the top of the east abutment and it was found that the east abutment bearing seat is tilting towards the river. The southeast wing wall is tilting towards the south.

The report speculates that the observed gap changes may be due to movement of the east abutment, due to the instability of the existing abutment wall and/or footing. This could not be stated conclusively without further investigation.

Pavement and Ground Structure

The boreholes taken behind the east abutment indicate that the pavement and ground structure consists of about 80 mm of asphalt pavement, 300 mm of granular road base, and silty clay and sandy silt overlying very strong gneiss bedrock at a depth of about 8 metres below the roadway surface. Groundwater was encountered in both boreholes at a depth of about 4.1 metres below the road surface.

The native soil under the east abutment was found to be very soft to soft clayey silt to silty clay. The report states that "based on the existing soil conditions, the observed tilting of the abutment wall has likely resulted from overstressing of the founding soils".

East Abutment Concrete

The vertical core taken through the entire height of the east abutment wall indicated that the height of the wall is about 3.5 metres. Two concrete samples from this core were tested for compressive strength and the measured strengths were 18.2 MPa and 14.2 MPa. A common result for new concrete in good condition would be 40 to 50 MPa. The MTO Structural Rehabilitation Manual classifies concrete weaker than 20 MPa as "poor quality" concrete.

The horizontal core was taken through a horizontal cold joint below the bearing seat and indicated that the cold joint is continuous through the thickness of the wall and the thickness of the wall at the corehole location was 1.7 metres. This indicates that the back of the abutment wall is inclined i.e. the wall is thicker at the bottom than at the top, where it measures 1.2 metres wide.

Recommendations for Rehabilitation

Underpinning or replacement of the east abutment is recommended due to the age of the abutment, the low concrete compressive strength measured, and the apparent abutment movements. Foundation measures supported on bedrock are recommended due to the relatively soft native soil. Driven piles are not recommended due to the need for heavy equipment and the resulting vibrations. Grouted micropiles socketed into bedrock are the recommended option for both underpinning of the existing abutment and construction of a new abutment. Helical piles are also recommended, but only for underpinning of the abutment.

4. LABORATORY TESTING

4.1 Concrete Testing

The Golder condition survey report for the investigation work performed at the east abutment, west abutment and pivot pier is included in Appendix H. A summary of the report is as follows. Measured chloride contents above the accepted threshold for de-passivation of embedded reinforcing steel were not measured in any cores.

East Abutment

There was a general absence of coarse aggregate in the cores from the east abutment, with most aggregate observed less than 10 mm in size.

The three cores tested for compressive strength measured 10.7, 17.4 and 12.5 MPa. Combined with the two results from the geotechnical investigation, the average measured concrete compressive strength from the east abutment is 14.6 MPa, below the 20 MPa MTO threshold for poor quality concrete.

West Abutment - North Wingwall

As indicated on the existing drawings, the west abutment was partially re-faced circa 1970. A single core from older concrete in the north wingwall of the abutment was tested for compressive strength and measured 10.8 MPa.

West Abutment Wall

Compressive strength testing of two cores from the newer concrete in the west abutment wall was performed and results of 44.1 and 38.3 MPa were obtained. The concrete was found to be high quality with well-proportioned coarse aggregate and reinforcing steel was encountered in both cores.

Pivot Pier

Compressive strength testing of three cores taken from the top of the pivot pier was performed and results of 23.5, 30.3 and 34.6 MPa were obtained. The concrete in the cores was found to be in good to fair condition, with signs of possible alkali-silica reaction.

East Pier

Compressive strength testing of two cores taken from the top of the east pier on the south side was performed and results of 17.0 and 23.9 MPa were obtained. The concrete in the cores was found to be in good to fair condition. The aggregate in the concrete was well dispersed with some aggregate particles having a maximum top size greater than 40mm.

4.2 Paint Testing

Six samples of the coatings were taken from the bridge and sent to Paracel Laboratories for lead and mercury content analysis. The Paracel analysis results report is included in Appendix F; a summary of the results is as follows:

Sample No.	Span	Location	Mercury Content (µg/g)	Lead Content (µg/g)
#1	Swing	North stringer, west end bay	<2	16700
#2	Fixed	T2S at U3S	<2	6710
#3	Swing	North truss, west bay	<2	64900
#4	Swing	East portal frame	<2	31800
#5	Fixed	FB2 at L2	<2	6060
#6	Fixed	FB5 at L5	<2	3690

Table 4 – Summary of Coating Testing Results

The reporting limit for mercury content for the purposes of the analysis was 2 µg/g, so essentially no mercury was detected in any of the samples.

The current Canadian Surface Coating Materials Regulations (SOR/2005-109) dated 14 November 2011, limits the concentration of total lead present in surface coating materials to 90 mg/kg (parts per million or ppm) or 0.009%. The lead content of the samples tested ranged from 3690 ppm (0.369%) to 64900 ppm (6.49%). The lead concentrations in the swing span coating samples were considerably higher than those from the fixed span. All samples tested contained concentrations of lead much higher than current acceptable limits for surface coatings.

Any blast-cleaning work on the spans for future re-coating would therefore have to take all measures necessary for lead abatement in accordance with current Ontario Ministry of Labour regulations and guidelines.

5. MECHANICAL INSPECTION

5.1 Inspection Findings

The following section is information prepared by SBE documenting the findings of their mechanical inspection of the swing span. The mechanical inspection photographs are included in Appendix I.

5.1.1 Span Support Machinery

The span is supported at its center by a center pivot. The wearing surfaces of the center pivot assembly are inaccessible without jacking the bridge and disassembling it. This work was not performed as part of the field inspection.

Six balance wheels are provided to stabilize the span during operation. Balance wheels are provided to accommodate minor imbalance in the structure and imbalance due to external loading including wind and ice loads. When properly adjusted, the balance wheels allow the span to tip slightly prior to the wheels coming into contact with a balance wheel track that is secured to the pier. The balance wheels are typically not designed to carry dead load or live load.

When the span is closed and open for vehicular traffic, the east end is supported by two castor wheels which bear on rest plates. The west end is supported by two hydraulic cylinders which extend to deflect the end of the span. The west end of the span also has two end castor wheels which no longer contact their rest plates but do provide a limited function in preventing excessive tipping of the span.

The following observations were made of the span support machinery components:

5.1.1.1 Center Pivot

The general external condition of the center pivot ranges from fair to poor. The following conditions were noted:

- The fasteners attaching the top plate to the structure are moderately corroded with moderate section loss (Photo M1);
- One of six anchor bolts exhibits corrosion and light section loss (Photo M2); and
- The pivot girder assembly directly above the center pivot collects debris with standing water present. The condition of the structural steel at this location is poor, with moderate to heavy section loss evident at the rivets (Photo M3).

The center pivot is oil-lubricated. Oil was present in the stand pipe for the bearing housing. No abnormal noises were noted during operation.

The wearing surfaces of the center pivot assembly are inaccessible without jacking the bridge and disassembling it. Maintenance personnel interviewed during the inspection have no record of internal inspection of the center pivot assembly ever being performed.

5.1.1.2 Balance Wheels/Rail

Clearances between the wheels and the rail were measured with the span closed and the end lifts engaged. Clearances range from 0 mm to a maximum of 3.6 mm. Measured clearances are as follows (wheels are numbered counter clockwise with north wheel starting at No. 1):

Wheel No.	Clearance
1	2.5
2	1.3
3	1.0
4	3.6
5	0.0
6	0.0

Table 5 – Wheel Clearances

At wheels No. 5 and 6, there is an impression on top of the rail from the wheel as a result of carrying live load (Photo M4). The balance wheels are not intended to carry live load.

The condition of the balance wheel rail is poor. The following conditions were noted:

- The rail and anchor bolts exhibit moderate corrosion and section loss;
- The rail support pier is undermined along a significant portion of the rail (Photo M5);
- The rail was observed to deflect under loads from the balance wheels during operation; and
- The rail is not flat and impressions are present where balance wheels #5 and #6 contact the rail, as noted above.

The balance wheel clevises are in fair condition. The assemblies and mounting bolts exhibit moderate corrosion and light section loss (Photo M6).

The balance wheel bearings are inaccessible for clearance measurements. However, clearance can be checked by rocking the wheel. Based on this indirect method of checking, some clearances appear to be in excess of an ANSI RC6 fit, which is the required fit for bearings of this type per CSA S6-06, the Canadian Highway Bridge Design Code (CHBDC).

Lubrication ports for the balance wheels are clogged. Maintenance personnel drip or spray oil on to the wheel/clevis interface, allowing oil to infiltrate the bearing.

5.1.1.3 East End Castor Wheels/Rest Plates

Slight movement was noted between both end rest plates and the pier. The south rest plate anchor bolt nuts exhibit gaps under the head indicating the anchor bolts are not properly tightened. One of the four north rest plate anchor bolts is bent (Photo M7).

The castor wheel bearings are inaccessible for clearance measurements. However, clearance can be checked by rocking the wheel. Based on this indirect method of checking, the clearances do not appear to be excessive.

The rest plates exhibit heavy wear due to contact with the rollers. No impact or movement was noted between the roller and rest plate, indicating that the wear may be due to over loaded components (Photo M7).

5.1.1.4 West End Castor Wheels/Rest Plates

There are two west end castors that are no longer utilized to support the swing span (Photo M8). The west end castors still provide a limited function of prevent excessive tipping of the span as it reaches the closed position.

The northwest end castor rest plate also serves as an end stop and is commented on in the following section.

5.1.1.5 End Lift Jacks

The end lift jacks directly lift the end of the span and are not self locking so that hydraulic pressure must be maintained in the end jack driving machinery in order to maintain the load. As such, the arrangement of the end lift jacks does not meet the requirement of the CHBDC that the end lift "actuating mechanism shall be non-reversible under the action of the live load." Failure of the hydraulic piping system (which has occurred previously according to maintenance personnel) would result in failure of the end jacks to support live load. This is a safety concern.

The external condition of the end lift jacks is fair. The cylinder bodies, base plates, and anchor bolts exhibit light corrosion (Photo M8). The cylinder rod and rod seals are in good condition.

The measured lift height was 20.6 mm at the south end lift and 19.1 mm at the north end lift. Based on the measured gaps at the east end castors prior to raising the end lift jacks it is estimated that of this lift height approximately 6.4 mm results in tilting of the span and the remainder is deflecting the span. Based on the behaviour of the span under live load the end lift deflection and resulting dead load reactions appear adequate.

Both end lift cylinder base plates anchor bolts are not properly tightened (Photo M9).

5.1.2 Locking Pin Machinery and End of Travel Stops

End of travel stops are provided at the northwest corner and the southeast corner of the span to limit the range of travel as the span approaches the closed position. No end of travel stop is provided in the open position. Two locking pins are provided. The west locking pin is a hydraulically actuated locking pin mounted to the southwest approach pier. The pin is hydraulically released in order to swing the bridge open. Once clear of the receiver the hydraulic pressure is released and a spring pushes the pin back to the extended position so that as the span reaches the closed position the pin automatically engages its' receiver which is mounted on the southwest corner of the swing span. The east locking pin has a similar arrangement but is manually released via a lever located at the center of the bridge at the north truss. The east locking pin is located at the bridge centerline at the east end of the swing span. The receiver is mounted on the east approach pier. The following observations were made.

5.1.2.1 West Locking Pin

The external condition of the west locking pin machinery is fair. All components and the structural steel that supports the locking pin exhibit light corrosion (Photo M10).

The design of the locking pin machinery does not provide for energy absorption. When the locking pin engages, there is an impact load depending on the speed at which the span reaches the fully closed position. The impact loads have resulted in heavy wear of the lock bar guides. It is common, where automatically engaging locking pins are provided for swing bridges, to provide a spring loaded receiver to mitigate impact loads.

5.1.2.2 East Locking Pin

The operator reports that the east locking pin is disengaged each morning and engaged each night. When the east locking pin was engaged and disengaged during the inspection it was found that the locking pin does not travel far enough to engage the receiver (e.g. it was disengaged at all times regardless of the position of the actuating lever). See Photo M11.

The external condition of the locking pin machinery is fair. All of the components exhibit corrosion (Photo M12).

5.1.2.3 East End of Travel Stop

The end of travel stop is installed with an energy absorbing pad that is in poor condition (Photo M13).

The end of travel stop anchor bolts are in poor condition and exhibit evidence that the stop was impacted resulting in the anchors being slightly pulled out (Photo M14).

5.1.2.4 West End of Travel Stop

The rest plate anchor bolt heads are cut off and do not secure the rest plate to the pier (Photo M15). The integrity of the stop is poor as the rest plate can be lifted out of place.

The end of travel stop is not provided with an energy absorbing pad, however there is no evidence of contact indicating the span stops within the limits of the locking pin and east end of travel stop.

5.1.3 Span Drive Machinery and Hydraulic Power Unit

The span is provided with a single hydraulic power unit (HPU) that operates both the end lift jack cylinders and a pair of slewing cylinders that operate the span. The bridge was originally equipped with electrically operated gear drive machinery. The current hydraulic system was rehabilitated in 2008. The HPU is housed in a separate building adjacent to the swing span and is well protected from the elements. Flexible hoses, buried underground in conduit, connect the HPU to rigid piping at the pier. The final connection between the rigid piping and the span drive cylinders is made with a short run of flexible hose. The final connections at the end lift jacks are with flexible hose. The following observations were made.

5.1.3.1 Hydraulic Power Units / Operation

The HPU's are in generally good external condition. No leakage or significant corrosion was noted.

Operating pressures were monitored by observing the pressure gages provided at the hose connection at the HPU. During operation of the span, the system pressure is 1,250 psi. There are no means provided to measure cylinder pressures.

The HPU and control system provides for semi-automatic control defined as follows: The fluid flow automatically increases from zero to normal volume and back to zero again for span acceleration and deceleration by the single operation of a hand lever. The HPU only provides for single speed operation and there is no ability to operate the span at a constant, reduced speed. The behaviour of the span as it approaches the fully closed position is therefore dependent upon the skill of the operator.

Based on the behaviour of the span during operation, there are no brakes or equivalent hydraulic devices (e.g. counterbalance valves) provided to hold the span stationary or allow for motion control as required by the CHBDC. However, a hydraulic schematic was not provided for the system to confirm this statement.

5.1.3.2 Hydraulic Hose / Piping

The condition of the piping and flexible hoses at the HPU is generally good with the only noted deficiency being that one hose is abraded at a location near the hose exit from the operator's house (Photo M16).

The condition of the flexible hoses at the span drive cylinders is poor. The hoses exhibit abrasions from contact with the center pier (Photo M17). In addition, the south cylinder blind end hose is nicked and exhibits blistering (Photo M18).

The blind end flexible hose connecting the HPU to the piping at the pier exhibits a severe bend radius (less than 120 mm radius) and should be adjusted to eliminate the severe bend radius (Photo M19).

The hydraulic piping to the end jacks is in good condition.

The west locking pin hydraulic cylinder blind end hose is damaged (Photo M20).

5.1.3.3 Span Drive Cylinders

The general external condition of the span drive cylinders ranges from good to fair. The following conditions were noted:

- The external condition of the span drive cylinders is good;
- The cylinder rods are in good condition. No scoring was observed;
- The cylinder rod seals are in good condition. No significant leakage was observed;
- Both cylinder pin connections are equipped with lubrication fittings and hoses to facilitate lubrication. All lubrication was found to be recent and adequate;
- The blind end clevis and bracket for both cylinders range from good to fair condition. The blind end clevis and bracket for the cylinders collect debris and exhibit light corrosion (Photo M21); and
- The rod end clevis brackets for both cylinders are in good condition.

5.1.4 Traffic Gate Machinery

The span is provided with a two traffic warning gates. The gates are of an obsolete standard commercial design. The following observations were made.

- The gate housings are in fair condition (Photo M22);
- The gate arm bearings are in poor condition and appear heavily worn (Photo M23); and
- The gates operated adequately during the inspection however the units are aged and of an obsolete design as evinced by the need to make a custom replacement brake shoe.

6. ELECTRICAL INSPECTION

6.1 Inspection Findings

The following section is information prepared by SBE documenting the findings of their electrical inspection of the swing span. The electrical inspection photographs are included in Appendix J.

6.1.1 Electric Utility Service

The electric utility service to the bridge is derived from an overhead 3 phase, 4 wire medium voltage distribution system. A single phase service has been tapped from the medium voltage distribution service to feed a single phase, oil filled, pole mounted 50kVA transformer that provides a 120/240 volt single phase service to the bridge and the residential customers in the neighbourhood at the west approach. From the utility service transformers the service feeder runs overhead and from a local pole, down a conduit to the utility metering equipment located outside of the bridge control building. The utility feeder is then run through the wall of the control building to terminate in the electrical panel board located at the lower level of the bridge control building. The bridge operating devices are fed from the circuit breakers in the electrical panelboard (See Photo E2)

The primary side of the transformer is provided with both a fused cut-out and lightning arrestor for transformer protection. The electric utility service equipment appears in good condition with no sign of corrosion or discolouration through overheating (See Photo E1).

The electric service voltage was measured under no-load condition to determine the adequacy and stability of the bridge electric utility service.

Item	Description	Voltage
1	Phase 1-to-Phase 2	243.4 Volts
2	Phase 1-to-Ground	121.6 Volts
3	Phase 2-to-Ground	121.6 Volts

Table 6 – Electric Service Voltage

The measurements of voltage were taken with all bridge auxiliaries operational, but the bridge drive system, including traffic gates switched off. From the above, it can be seen that the no-load phase voltages are balanced. And the mean phase to phase voltage is within 1.5% of the nominal voltage of 240 volt. This is an indication that the incoming voltage is stable.

The only deficiency noted for the bridge utility service is that the bridge is not provided with a standby power or auxiliary means of operating the bridge in the event of power outage.

6.1.2 Bridge Operating Electrical System

6.1.2.1 Main Swing Span Hydraulic Pump Motor

The bridge swing span is hydraulically operated from a hydraulic power unit (HPU) in the bridge control building. A single hydraulic pump motor pressurizes the hydraulic system for the swing operation. Note this hydraulic system is only used to drive the span and no other auxiliary drives; see below for bridge end lift HPU. The swing span HPU pump motor is a single phase 230 volt, 7.5 HP motor manufactured by WEG and is controlled from a starter housed in the wall mounted enclosure near the HPU (Photo E3).

Motor nameplate data was collected and recorded as follows:

Specification	**Swing Span HPU Pump Motor**
Manufacturer:	WEG
VJP Part No.:	TC010104
S/N:	E615336
Type:	EM
PF:	0.97
Rating:	IP55
Frame:	215TC
Phase:	1
Hz:	60
Volts:	208-230
Amps:	44.8-39.0
Horsepower:	7.5
Duty:	Cont.
Speed:	1730 RPM

The pump motor and motor starter within the wall mounted enclosure were replaced in the recent past and all are in good new and operational condition (Photos E4 and E5). A spare swing span HPU pump motor is provided in a close vicinity of the HPU.

In an effort to determine the operating characteristic of the swing span HPU pump motor its operating load characteristics (voltage, current and kW) were measured and recorded (See Figures 2 and 3 below).

Figure 2: Voltage and current parameters for the swing span pump motor.

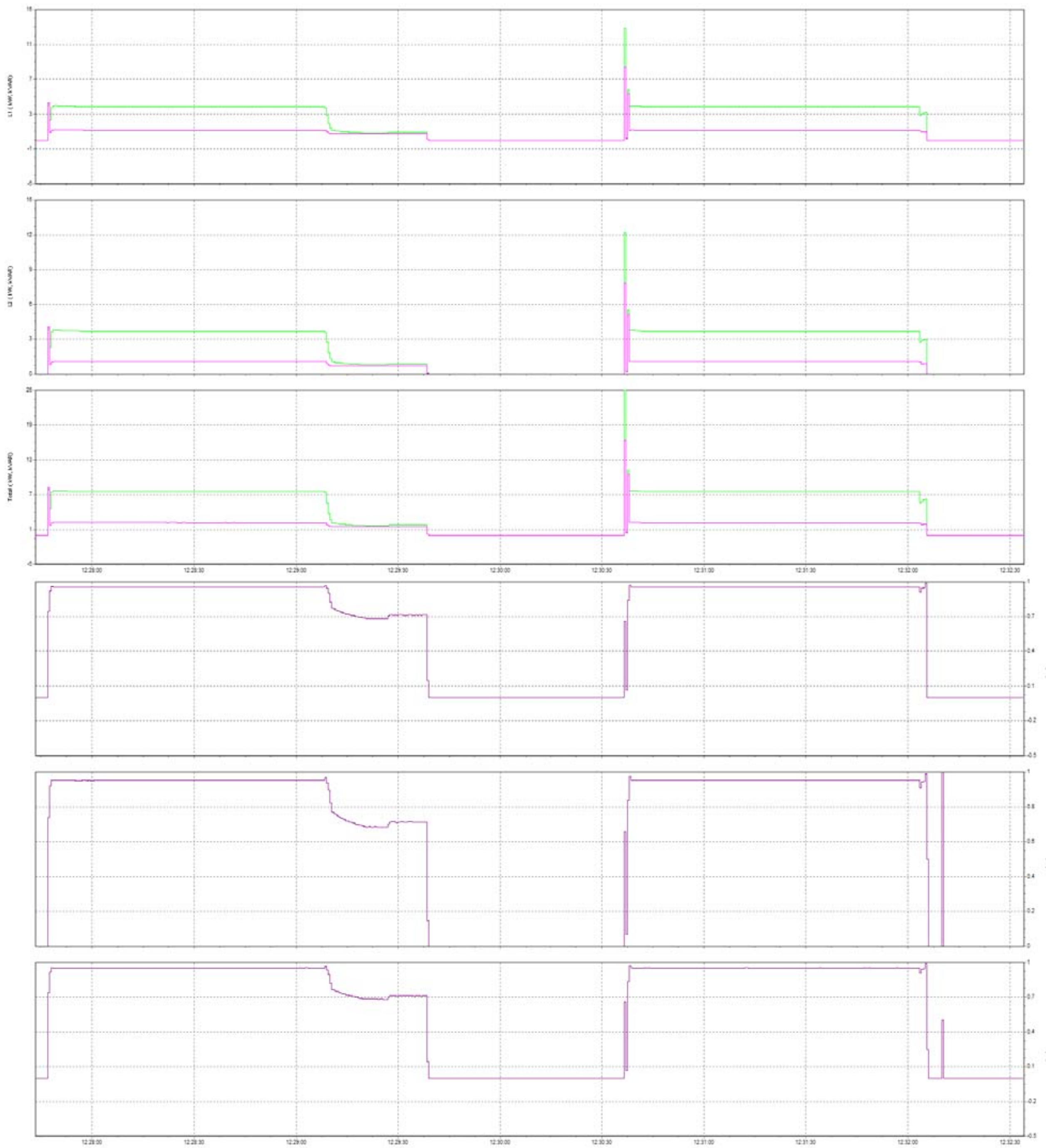

Figure 3: Power parameters for the swing span HPU pump motor.

From the above results, it can be seen that the average load currents draw of the swing span hydraulic pump motor during the bridge operating cycle is within the full load current indicated on the motor nameplate which is 44.8-39.0 A. This is an indication that the rating of the main pump motor is operating close to its full load output. The average power output recorded from the pump motor is approximately 5.5 kW which equates to 7.4 hp which once again is approaching the full load output of the pump motor. This is an indication that the HPU is somewhat undersized to cope with overload and transient load conditions. The electrical surges recorded on the HPU pump motor are minimal during bridge operation which is an indication that the loading of the bridge operating system is uniform throughout its operating cycle. Additionally, the recorded average power factor for the pump motor is 0.95 which is consistent with the motor nameplate data and is consistent with this type of single phase motor.

Deficiencies noted for the main swing span hydraulic pump motor are described as follows:

- Based on the chart recordings the swing span HPU appears undersized for the prevailing duty and under severe loading conditions such as wind loading could cause a pump motor trip;
- Swing span HPU motor power junction box is not sealed (Photo E4); and
- The starter overload reset pushbutton cannot be triggered by pressing the reset button on the enclosure cover as the starter enclosure is that of original installation during the 1991/1992 rehabilitation. The installed starter and overload unit is a replacement of the original starter.

6.1.2.2 PLC Controller

The field feedback devices and operator's control station operating and indication devices are connected to the PLC controller located in a wall mounted enclosure at the roadway level in the bridge control building for control and interlocking of the entire bridge operation. The PLC controller is of an obsolete type with spare parts unavailable. This PLC should be upgraded to a modern PLC where spare parts can be easily obtained (Photo E6). When the bridge operator commands the bridge to open or close from the control station, the PLC controller starts the sequence of operation described below.

1. Press and hold 'OPEN' button.
    a. Traffic signal will turn 'red'
    b. Gates lowering → fully lowered
    c. Wedges pulling → fully pulled
    d. End lift retracting → fully retracted
    e. Bridge swings at full speed → hydraulic motor de-energized by the fully open limit switch (Locking Pin Extended)
2. Press and hold 'CLOSE' button.
    a. Bridge swings close at full speed
    b. At nearly closed position the bridge operation automatic reduced to creep speed
    c. From nearly closed position the bridge swung at creep speed to fully closed position
    d. Fully closed → Locking pin locked in place
    e. Raise end lifts → fully raised
    f. Raise gates → fully raised
    g. Traffic signal turns 'amber'

Note: Operation can be stopped by release the 'OPEN/CLOSE' pushbutton or by the emergency stop pushbutton.

Operation of the swing span is under the sequenced control of the PLC. Movable span cannot be swung until the necessary sequence operation is completed.

Deficiency noted for the PLC controller described as follows:

- The bridge PLC controller is obsolete and it is difficult to obtain spare part in the event of a PLC controller component failure.

6.1.2.3 Relay/Contactor Panel

The relay/contractor panel is located in the HPU room on the lower level of the bridge control building (Photo E3). The control relays are all in good operating condition with all wires well tagged for ease of troubleshooting. The contactors are of the solid state type and used devices in the panel are generally in good operating condition.

Deficiencies noted for the relay/contactor panel are described as follows:

- The spare parts and spare wires were left at the bottom of the cabinet (Photo E18);
- One of the solid state contactor LED light is broken; and
- Spare or unused wires are not properly terminated.

6.1.3 Bridge Control Station

The operator's control station is located directly outside the operator's control building that is located southwest approach. The control station contains one (1) three position maintained switches, two (2) indication lights, two (2) pushbuttons, two (2) indicating pushbuttons, and an emergency stop pushbutton to facilitate operation of the swing span from this single location. Although exposed to the harsh environment, the control station is housed in a PVC enclosure and all devices appear in fair operational condition with only minor signs of deterioration (Photo E7). The control station provides very limited indication of the bridge status to the operator; the only indications provided for the operator at this location are indication of red light failure and hydraulic low oil, but does not provided status of the traffic control equipment, position of end lift devices, position of locking pin and position of the swing span. The control station is only provided with automatic sequenced operation for the bridge operating equipment.

The location of this control station affords the operator vision of the bridge approaches as well as the waterway to enable him to safely operate the bridge but does not provide him with line of sight vision of maintenance personnel performing maintenance on the swing span hydraulic system. The control station is not provided with a keyed 'On-Off' switch or means of de-energize the control station when the bridge is unmanned. This feature should be provided to prevent potential break-in to the control console and unintentional operation of the bridge by non-authorized personnel.

Deficiencies noted for the control station are described as follows:

- The control station does not provide any status indication for the traffic control devices and the bridge operating device include the movable span;
- One of the pushbuttons on the lower right corner of the control station is not labelled;
- No means of disconnecting the control station power when bridge is unmanned; and
- Individual gate-operating switch is not provided as per code.

6.1.4 Bridge End Lift System

The bridge is provided with two (2) hydraulically operated end lifts, one at each corner of the west end of the moving span (See Photo E8). A separate HPU independent from the swing span HPU is used to pressurize the hydraulic system for the operation of the bridge end lift system and the locking pin. This pump motor is a single phase 230 volt, 3HP motor manufactured by Leeson and is controlled from a starter housed in a separate wall mounted enclosure near the hydraulic unit (Photo E3).

Motor nameplate data was collected and recorded as follows:

Specification	**Main Span Drive Motor**
Manufacturer:	LEESON
Cat. No.:	131533MOO
Model:	C184C17FB12C
Frame:	NX184TC
Phase:	1
Hz:	60
Volts:	208-230
Amps:	16.8
Horsepower:	3
Duty:	Cont.
Speed:	1740 RPM

Operation of the bridge end lifts is under the sequenced control of the PLC. Bridge end lifts can also be manually operated for maintenance and troubleshooting purposes. An end lift local control panel is provided in the HPU room with an Auto/Manual control switch allowing manual operating of the end lifts (Photo E9). The pump motor starter is housed within a wall mounted enclosure in the HPU room and both the pump motor and motor starter were replaced in the recent past and all are in good operational and new condition (Photos E10 and E3). Spare end lift and locking pin HPU pump motor is provided in close vicinity of the HPU.

In an effort to determine the operating characteristic of the spanned lift and locking pin HPU pump motor its operating load characteristics (voltage, current and kW) were measured and recorded (See Figures 4 and 5 below).

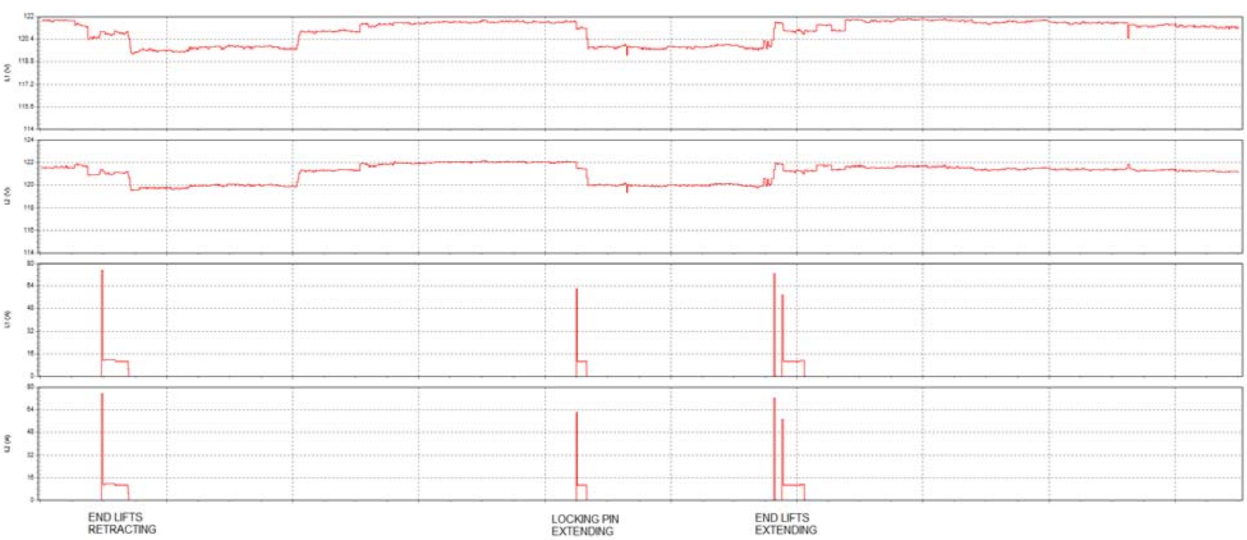

Figure 4: Voltage and current parameters for the end lift and locking pin hydraulic pump motor.

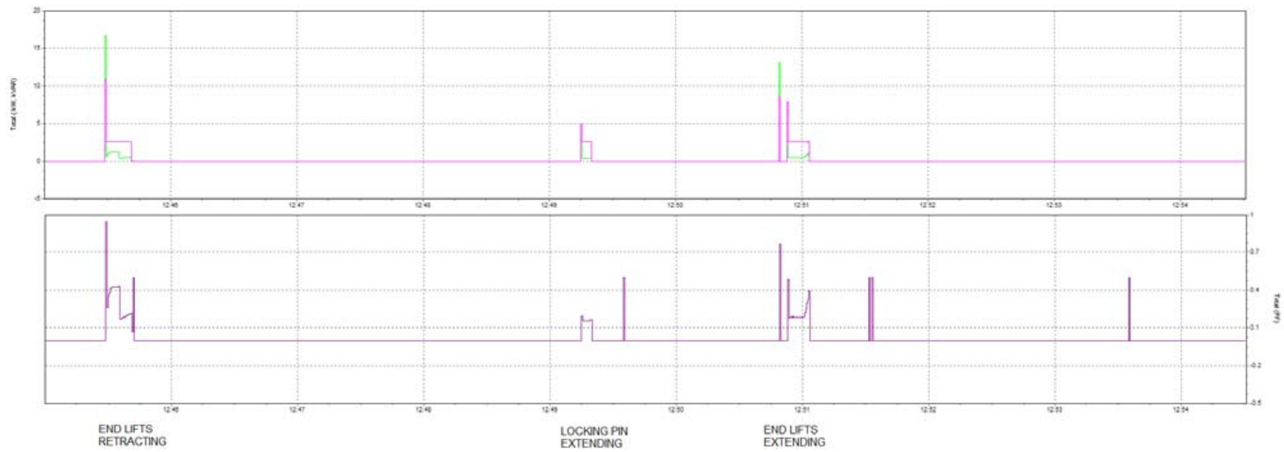

Figure 5: Power parameters for the end lift and locking pin hydraulic pump motor.

From the above results, it can be seen that the average load currents of the end lift and locking pin hydraulic pump motor are within the full load current indicated on the motor nameplate which is 16.8 A. This is an indication that the rating of the pump motor and HPU has been correctly sized. The electrical surges recorded on the end lift and locking pin HPU pump motor are minimal during bridge operation which is an indication that the loading of the end lift and locking pin operating system is uniform throughout its operating cycle.

Each end lifts are provided with end lift extended and end lift retracted limit switches used for end of travel control. These limit switches are of the roller arm type manufactured by Cutler Hammer and are all in fair to good operational condition (Photo E8). Maintenance staff indicated that the end lifts are covered as part of their wintering procedure to protect the equipment.

Deficiencies noted for the bridge end lift system are described as follows:

- The limit switch support steel plate is heavily corroded (Photo E8);
- Fittings for the Teck cable used to connect the limit switches show initial signs of corrosion;
- Fittings for the hydraulic hosts show initial signs of corrosion but this item is addressed in more detail in the Mechanical section of the report; and
- Roller arm for the limit switches shows minor signs of corrosion.

6.1.5 Locking Pins

The moving span is provided with two locking pins that locks the span in the fully closed position. The east locking pin is hydraulically operated and is withdrawn with the use of a hydraulic cylinder that pulls the pin and charges a spring (Photo E11). When the locking pin engages, there is an impact load depending on the speed at which the span reaches the fully closed position. The east locking pin is in operating condition with minor sign of corrosion. The east locking pin is operated with the same hydraulic system as the one for the bridge end lift system. Refer to bridge end lift system for description of electrical operating characteristic for the hydraulic pump.

The east locking pin is provided with the pin extended and pin retracted limit switches used for end of travel control. These limit switches are of the roller arm type manufactured by Cutler Hammer are all in fair to good operational condition (Photo E11).

The moving span is also provided with a second locking pin on the east end of the span. This locking pin is manually operated and is only engaged overnight to lock the bridge in place. Refer to the mechanical section of the report for evaluation of the east locking pin.

Deficiencies noted for the locking pin system are described as follows:

- The locking pin mounting plate is moderately corroded and not firmly mounted (Photo E11);
- Fittings for the Teck cable show initial signs of corrosion;
- Limit switch arms shows initial signs of corrosion; and
- No safety limit switches were provided for the east locking pin to prevent operation of the bridge when the pin is extended.

6.1.6 Bridge Drive System Control Limit Switches

The bridge drive system is provided with end of travel and safety interlock limit switches (nearly closed, fully closed and fully open). The limit switches are of the roller arm type manufactured by Cutler Hammer. Normally the operator will manually release the control pushbutton to stop the swing span HPU pump motor before the full closed or full open limit switch is hit to prevent bridge slamming. In the event that the operator fails to manually stop the bridge, these limit switches perform the intended function of de-energizing the HPU pump motor and stopping the bridge. The nearly closed limit switch is used to swing the bridge in creep speed from nearly closed position to fully closed position. The limit switches were all found to be operational at the time of inspection (Photo E12).

Deficiencies noted for the bridge drive system control limit switches are described as follows:

- The nearly closed and fully closed limit switch is covered with some debris (Photo E12); and
- Moderate corrosion is observed for the Teck cable fitting as they are exposed to the harsh environment.

6.1.7 Vehicular and Marine Traffic Control

The Traffic Control group contains the Traffic Lights, Roadway Gates and Aids to Navigation.

6.1.7.1 Traffic Signals and Signs

The traffic light installation consists of one (1) three section light fixture vertically mounted on a pole at each approach to the bridge (See Photo E13). Although the traffic signals consist of a Red, Amber and Green section, only the Amber and Red section is being used for the single lane roadway over the bridge. The light turns to Red during a bridge operation and flashing Amber when open for vehicular traffic.

Both bridge approaches are provided with a warning gong to provide audible warning for the vehicular and pedestrian traffic (Photo E15). The warning gongs are mounted on top of the gate housing.

The only deficiency noted for the traffic signals and signs is noted as follows:

- The west Stop bar is worn (Photo E14).

6.1.7.2 Gates

The bridge is provided with two (2) traffic gates, one for each approach (Photo E15). The gate motor contactors are located in the relay panel in the HPU room. The gates are also provided with two roller arm type limit switches, one for gate raised and one for lowered (See Photo E16). The limit switches are of Cutler Hammer manufacture and appeared to be in satisfactory condition at the time of inspection.

Gate Specification is recorded as follows:

Manufacturer:	Western Railroad Supply Co.
Serial No.:	1996
Phase:	1
Hz:	60
Volts:	220

The following deficiencies were noted for the traffic gates:

- The gates are not provided with safety interlock door or hand crank limit switches;
- The paint on the gate housing is peeling off and the housing shows signs of corrosion;
- Due to its age and obsolete design, consideration should be given to replacing these gates to ensure future reliability of gate operation; and
- The gate housing doors were not provided with gaskets to maintain its weatherproof integrity for the equipment inside the gate housing.

6.1.7.3 Aids to Navigation

The bridge navigational signals consist of two (2) single navigation signal (one red and one blue) mounted facing the south side of the channel and one (1) single red navigation signal mounted facing north side of the channel (Photo E17). These lights are provided to give marine traffic indication of the bridge status. The blue light goes on only when the bridge is fully open. The red signal is on at all other times. The navigation lights were working accordingly at the time of inspection.

The following deficiency was noted:

- The bridge is not provided with any fender navigation lights as per Coast Guard requirements.

6.1.8 Cables, Junction Boxes and Submarine Cable

The bridge outdoor electrical installation consists of numerous Teck cables and junction boxes and submarine cable junction boxes.

The bridge electrical system uses Teck type cables running underground, underwater and the fixed and movable structure to feed all electrical devices throughout the bridge. Generally the Teck cables appear to be in fair physical condition.

The bridge is provided with interior and exterior junction boxes. The interior junction boxes appear to be in good condition. Some of the exterior junction boxes have corrosion and debris inside. The bridge is provided with a submarine cable junction box located on the centre pier. The submarine cables are also used to feed traffic gate and traffic signals on the east approach. The submarine junction boxes appear to be in good physical condition and their weatherproof integrity appears to have been maintained (Photo E19). The submarine cables provided are of the Teck cable type.

The following deficiencies were noted for the general cable and junction box installation and condition:

- Spare/unused wires are generally not properly terminated in enclosures or junction boxes (Photo E20); and
- Submarine cable junction box has loop of wires lying on top of terminal blocks and some wires were not terminated appropriately (Photo E19).

6.1.9 Lighting

Generally the bridge facilities are provided with sufficient lighting for maintenance or troubleshooting.

Deficiency noted for the bridge lighting is noted as follows:

- Bridge facilities are not provided with emergency lighting or exit signs as per safety code.

7. LOAD EVALUATION - SWING SPAN

7.1 Evaluation Methodology

The load evaluation of the swing span was performed in accordance with Section 14 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06, which specifies methods for evaluating existing bridges.

The bridge was evaluated at Ultimate Limit States (ULS) only, as no significant deformation, vibration, or other Serviceability Limit State (SLS) issues have been identified, nor were any observed during inspection. The bridge was not evaluated for Fatigue Limit States (FLS). According to Section 14, a fatigue evaluation is necessary where there are fatigue-prone details or physical evidence of fatigue-related defects. The riveted connections in this structure are not considered fatigue-prone details. In addition, no physical evidence of fatigue-related defects was observed.

The live load distribution was carried out using a "Sophisticated Method", with a three-dimensional computer model of the span carried out in the analysis software package MIDAS by the MIDAS Information Technology Company Limited. A three-dimensional model of the bridge was created and applicable loads were applied. Member sizes were confirmed with field-measured dimensions. Diagrams of the model geometry can be found in Appendix K.

As no original structural drawings specifying structural steel grade are available, the yield strength and tensile strengths of the main structural steel members were taken as 210 MPa and 420 MPa, respectively, per Section 14 of the CHBDC. The rivet tensile strength was taken to be 320 MPa, per Section 14 of the CHBDC. The timber deck was assumed to be S-P-F (Spruce-Pine-Fir) No. 1/No. 2 grade, with a specified bending strength of 8.4 MPa.

Properties for the various sections were calculated using standard methods, assuming rectangular shapes for the component shapes. Section losses due to corrosion were accounted for by reducing the thicknesses of the component plates, angles, webs and other shapes in accordance with the measured remaining thicknesses of sound metal measured by ultrasonic testing. No allowance was made in the estimated section losses for future deterioration.

7.2 Loads

The structure weights (dead loads) were computed based on the original unreduced sections, the geometry of the bridge, and the material densities specified in the CHBDC.

The evaluation was performed to Evaluation Levels 1, 2, and 3. The loading that corresponds to Level 1 in accordance with the CHBDC is the CL1-625-ONT Truck and CL1-625-ONT Lane Load. The CL2-625-ONT Truck and CL2-625-ONT Lane Load correspond to Level 2, and the CL3-625-ONT Truck and CL3-625-ONT Lane Load correspond to Level 3, as per the CHBDC.

In the absence of traffic data for the roadway, a Class C Highway was assumed for the analysis, resulting in a Lane Load of 7 kN/m. A Class C Highway has an Average Daily Traffic (ADT) per lane of between 100 and 1000 vehicles, and an Average Daily Truck Traffic (AADT) per lane of between 50 and 250 trucks. During the inspection, the bridge operator reported traffic counts across the bridge of between 400 and 500 vehicles per day.

In accordance with Clause 14.9.5.4 of the CHBDC, temperature effects were not considered for this generally ductile structure. Wind loads and seismic loads were not included.

7.2.1 Structural Analysis

The determination of load factors in accordance with Section 14 depends on the target reliability index (β) for each member, which in turn depends on the system behaviour, element behaviour, and inspection level. The inspection level was Inspection Level 3, where the evaluator has directed the inspection of all critical and substandard components and final evaluation calculations account for the information obtained during the inspection.

The element behaviour of the loading girder, pivot girders, pivot diaphragms, floor beams, stringers and timber deck for moment and shear failure was taken as Category E3, where members are expected to fail gradually with noticeable deformation prior to failure. The element behaviour of truss members in compression is Category E1, as the members would be subject to rapid loss of capacity due to buckling, with little or no warning. The element behaviour of truss members in tension, including the bottom chord, is Category E3, as the members would generally be expected to fail gradually with noticeable deformation prior to failure. Element behaviour of connections is Category E1.

The system behaviour category varies with different member types. The top chords and bottom chords were classified as Category S1, where element failure leads to total collapse. The truss diagonals, truss verticals and the floor beams were classified as Category S2, where element failure probably will not lead to total collapse. The stringers and deck were classified as Category S3, where element failure leads to local failure only.

The target reliability indexes and dead and live load factors were then calculated according to Section 14. Table 7 summarizes the Target Reliability Indexes and resulting load factors used for the different member types. "D1" represent dead loads of factory produced components such as structural steel and truss members and "D2" represent dead load of timber deck.

Member Type	Shear / Moment / Tension				Bearing / Compression			
	Target Reliability Index (β)	Dead Load Factors, α_D		Live Load Factor, α_L	Target Reliability Index (β)	Dead Load Factors, α_D		Live Load Factor, α_L
		D1	D2			D1	D2	
Loading girder	3.00	1.07	1.14	1.49	---	---	---	---
Pivot girders	3.00	1.07	1.14	1.49	3.75	1.10	1.20	1.70
Floor beams	2.75	1.06	1.12	1.42	---	---	---	---
Stringers	2.50	1.05	1.10	1.35	---	---	---	---
Deck	2.50	1.05	1.10	1.35	---	---	---	---
Top chords	---	---	---	---	3.75	1.10	1.20	1.70
Bottom chords	3.00	1.07	1.14	1.49	3.75	1.10	1.20	1.70
Verticals	2.75	1.06	1.12	1.42	3.50	1.09	1.18	1.63
Diagonals	2.75	1.06	1.12	1.42	3.50	1.09	1.18	1.63

Table 7 – Summary of Target Reliability Indexes and Load Factors for Swing Span

Live load capacity factors (F) were calculated for the various members of the structure using the following formulation from Clause 14.15.2.1 of the CHBDC.

$$F = \frac{UR_r - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L(1+I)}$$

The member and connection resistances, R_r, were calculated based on Sections 9, 10, and 14 of the CHBDC. The resistance adjustment factor, U, was set to 1.0 in all cases, conservatively, as most members of the bridge exhibit some level of deterioration (per Clause 14.14.2 of the CHBDC). The dynamic load allowance, I, was automatically included in the MIDAS traffic loadings. No additional loads (A) were used in the analysis.

The bridge was evaluated in the swung closed, open to traffic position only, and the no additional jacking loads were considered at the east pier or west abutment. It was assumed that the hydraulic jacks at the west abutment only serve to lift the dead load stresses (the end sag of the span due to its own weight) out of the trusses, not to introduce any positive "pre-stress" into the structure.

7.3 Evaluation Results

Table 8 summarizes the minimum live load capacity factors calculated for the different member types.

Member Type	Minimum Live Load Capacity Factor, F		
	Evaluation Level 1	Evaluation Level 2	Evaluation Level 3
Loading girders	3.63	4.00	5.02
Pivot girders	**0.71**	0.79	1.01
Floor beams	0.97	0.97	0.97
Stringers	1.40	1.40	1.40
Deck	0.77	**0.77**	**0.77**
Top chords	1.46	1.63	2.08
Bottom chords	3.29	3.65	4.65
Verticals	2.46	2.72	3.46
Diagonals	1.78	1.96	2.48

Table 8 – Minimum Live Load Capacity Factors for Swing Span

A live load capacity factor of 1.0 indicates that the member is loaded to full capacity and cannot carry any additional load. A live load capacity factor less than 1.0 indicates that the member is loaded beyond capacity under the current CHBDC design vehicle loads.

As can be seen in Table 8, the pivot girders were found to be the most critical elements, with a minimum F of 0.71 for Evaluation Level 1, followed by the timber deck with an F of 0.77 for all evaluation levels. The critical failure mode for both types of members is bending.

The results of the evaluation indicate that the span may be triple load-posted as follows:

Evaluation Level	Weight Limit (tonnes)
1	43
2	34
3	19

Table 9 – Weight Limits for Swing Span

Triple load posting in accordance with the CHBDC means providing posting signs at each end of the bridge showing representations of the following three types of vehicle, along with the corresponding gross vehicle weight ratings (GVWR):

- Single unit vehicle with GVWR of 19 tonnes;
- Two-unit vehicle with GVWR of 34 tonnes; and
- Vehicle train with GVWR of 43 tonnes.

However, as detailed below, the fixed span load rating is lower than the swing span and therefore currently governs the load rating of the crossing.

8. LOAD EVALUATION - FIXED SPAN

8.1 Evaluation Methodology

The evaluation methodology used was generally the same as for the swing span, except as noted below.

The bridge was evaluated at Ultimate Limit States (ULS) only, and not for Fatigue Limit States (FLS). According to Section 14, a fatigue evaluation is necessary where there are fatigue-prone details or physical evidence of fatigue-related defects. The riveted connections in this structure are not considered fatigue-prone details. The square bar bottom chords, bottom chord I-bars and square bar hangers for the floor beams can be considered fatigue-prone details, but were examined during the inspection and no physical evidence of fatigue-related defects were observed.

The bottom chords in the east panels of the trusses, at the locations of emergency repair with steel cables, were assumed to be at full strength i.e. temporarily strengthened or replaced.

8.1.1 Loads

The loads used in the evaluation were calculated in generally the same fashion as for the swing span.

8.1.2 Structural Analysis

The structural analysis was carried out similarly to the swing span, except as noted below.

The element behaviour of the floor beams, stringers and wood deck for moment and shear checks was taken as Category E3, as the members would generally be expected to fail gradually with noticeable deformation prior to failure. The element behaviour of truss members in compression, and tension members connected by eye-bars such as the bottom chords and truss diagonals, is Category E1, as the members would be subject to rapid loss of capacity with little or no warning. The element behaviour of other tension members is Category E3, as the members would generally be expected to fail gradually with noticeable deformation prior to failure.

The system behaviour category varies with different member types: the top chords and bottom chords were classified as Category S1, where element failure leads to total collapse. The floor beams, truss verticals, and truss diagonals were classified as Category S2, where element failure probably will not lead to total collapse. The stringers and deck were classified as Category S3, where element failure leads to local failure only.

The target reliability indexes and dead and live load factors were then calculated according to Section 14. Table 10 summarizes the Target Reliability Indexes and resulting load factors used for the different member types. “D1” represent dead loads of factory-produced components such as structural steel and truss members and “D2” represents the dead load of the timber deck.

Member Type	Shear / Moment / Tension				Compression			
	Target Reliability Index (β)	Dead Load Factors, α_D		Live Load Factor, α_L	Target Reliability Index (β)	Dead Load Factors, α_D		Live Load Factor, α_L
		D1	D2			D1	D2	
Floor beams	2.75	1.06	1.12	1.42	---	---	---	---
Stringers	2.50	1.05	1.10	1.35	---	---	---	---
Deck	2.50	1.05	1.10	1.35	---	---	---	---
Top chords	---	---	---	---	3.75	1.10	1.20	1.70
Bottom chords	3.75	1.10	1.20	1.50	---	---	---	---
Verticals	2.75	1.06	1.12	1.30	3.50	1.09	1.18	1.45
Diagonals	3.50	1.09	1.18	1.45	---	---	---	---

Table 10 – Summary of Target Reliability Indexes and Load Factors for Fixed Span

Live load capacity factors (F) were calculated for the various members of the structure using the following formulation from Clause 14.15.2.1 of the CHBDC.

$$F = \frac{UR_r - \sum \alpha_D D - \sum \alpha_A A}{\alpha_L L(1+I)}$$

The member and connection resistances, R_r, were calculated based on Sections 9, 10, and 14 of the CHBDC. The resistance adjustment factor, U, was set to 1.0 in all cases, conservatively, as most members of the bridge exhibit some level of deterioration (per Clause 14.14.2 of the CHBDC). The dynamic load allowance, I, was automatically included in the MIDAS traffic loadings. No additional loads, A, were used the analysis.

In the structural modelling of the span, one unique characteristic of the span had to be accounted for. The welded connections of the stringers to the floor beams, to each other, and to the abutment and pier bearings means that the stringers are fully fixed across the span and share load with the truss bottom chords.

Bracing members were not rated as their required strength is controlled by non-live lateral loads such as wind and seismic and as such they do not affect the evaluation of the bridge.

8.2 Evaluation Results

Table 11 summarizes the minimum live load capacity factors calculated for the different member types, following replacement of the deteriorated east bottom chords.

Member Type	Minimum Live Load Capacity Factor, F		
	Evaluation Level 1	Evaluation Level 2	Evaluation Level 3
Floor beams	0.52	0.52	**0.52**
Stringers	0.67	0.67	0.67
Deck	0.77	0.77	0.77
Top chords	0.57	0.62	0.78
Bottom chords	**0.33**	**0.42**	0.54
Verticals	0.54	0.56	0.63
Diagonals	0.38	0.46	**0.52**

Table 11 – Minimum Live Load Capacity Factors for Fixed Span

The results indicate that the span is generally not capable of supporting the current CHBDC vehicle loads, which is not surprising given its age, condition, and the general slenderness of most of its members.

For Evaluation Levels 1 and 2, tension in the truss bottom chords was found to control the load rating, with F values of 0.33 and 0.42 respectively. At Evaluation Level 3, bending in the floor beams and tension in the truss diagonals was found to control, with an F of 0.52.

The results of the evaluation indicate that the span may be triple load-posted in accordance with the CHBDC as follows:

Evaluation Level	Weight Limit (tonnes)
1	19
2	18
3	12

Table 12 – Weight Limits for Fixed Span

Triple load posting in accordance with the CHBDC means providing posting signs at each end of the bridge showing representations of the following three types of vehicle, along with the corresponding gross vehicle weight ratings (GVWR):

- Single unit vehicle with GVWR of 12 tonnes;
- Two-unit vehicle with GVWR of 18 tonnes; and
- Vehicle train with GVWR of 19 tonnes.

8.3 Discussion and Limitations

It should be noted that the welded connections between the stringers and the floor beams, the welded connections at the east pier between the stringers and steel plate bearings, the stringer connections at the east abutment, and the welded connections joining the stringer ends together, create a situation wherein the stringers are behaving as tension ties between the east abutment and east pier, and are sharing tensile load with the truss bottom chords.

This continuity of the stringers across the span probably explains why the bridge has not shown more signs of distress when subjected to vehicle loadings higher than the current 3 tonne weight limit. According to reports from the bridge operator, such loads have occurred from time to time.

This continuity of the stringers may also explain the observed movements at the east abutment. The stringer configuration does not allow for any expansion or contraction of the stringers under temperature changes. The expected contraction of the stringers under a 40 degree C temperature drop from time of construction to mid-winter is about 15 mm. This contraction would introduce a high tensile force into the stringers which is likely one contributor to the observed movements in the east abutment. In addition, vehicle loads could cause the stringers to pull on the abutment in the direction of the centre of the span.

The attachment of the stringers to the east abutment (see Photo F31) was not verified by destructive testing during the inspection. This should be done if it is contemplated by PCA to raise the load limit for the bridge.

9. OVERALL BRIDGE RATINGS

Condition ratings were assigned for each main component group of the two bridge spans, in accordance with the 2010 BIM, and are shown on the inspection forms in Appendix A. These ratings were used to establish the Structural Condition Rating and Functional Rating of the two spans in accordance with the criteria outlined in the tables in Section 2.4 of the BIM.

9.1 Swing Span

Based on the results of the inspections, investigations and structural evaluation of the swing span, the current overall **Structural Condition Rating is 2 (Inadequate).** This rating is based on the bridge not meeting current CHBDC traffic loading and, even if load-posted in accordance with this report, being load-posted to more than 15% below CHBDC loading. CHBDC loading for Evaluation Level 1 corresponds to a vehicle weight of about 60 tonnes; according to the evaluation results the span could be posted to 43 tonnes for Level 1. These criteria are in accordance with the 2010 BIM.

The current **Functional Rating is 2 (Inadequate)**. This rating is also based on the allowable load posting being more than 15% below CHBDC loading, and on the lack of crash-tested bridge barriers on the span.

As Parks Canada has indicated that they may want to maintain the existing single load posting at the bridge of 3 tonnes, if the CHBDC live loading requirements are not taken into account, both the **Structural Condition Rating** and the **Functional Rating** are **2 (Inadequate)** as significant repairs are required to primary components.

9.2 Fixed Span

Based on the results of the inspections, investigations and structural evaluation of the fixed span, the current overall **Structural Condition Rating is 2 (Inadequate).** This rating is based on the bridge not meeting current CHBDC traffic loading and, even if load-posted in accordance with this report, being load-posted to more than 15% below CHBDC loading. According to the evaluation results the span could be posted to 19 tonnes for Level 1. These criteria are in accordance with the 2010 BIM.

The current **Functional Rating is 2 (Inadequate)**. This rating is also based on the allowable load posting being more than 15% below CHBDC loading, and on the lack of crash-tested bridge barriers on the span.

As Parks Canada has indicated that they may want to maintain the existing single load posting at the bridge of 3 tonnes, if the CHBDC live loading requirements are not taken into account, both the **Structural Condition Rating** and the **Functional Rating** are **2 (Inadequate)** as significant repairs are required to primary components.

10. RECOMMENDATIONS AND COST ESTIMATES

The following rehabilitation and maintenance work is recommended for the spans. The recommended work items have been prioritized for planning purposes. Refer to the BIM forms in Appendix A for descriptions of the defects recommended to be repaired.

Based on results of the visual detailed inspection, previous experience with similar projects, and published cost data, Class 'C' cost estimates of the recommended renewal works are provided below for budgetary purposes.

The cost estimates represent our opinion of probable cost for the proposed works, do not include taxes, and are in 2011 dollars. It is assumed that the items included under the same heading are performed in one contract, so overhead costs such as mobilization and traffic control are distributed over those items. Maintenance work to be performed by PCA forces has not been cost-estimated.

10.1 Swing Span

10.1.1 Structural

<u>Immediate Remedial/Maintenance Work for Safety Reasons</u>

1. No immediate remedial/maintenance work is recommended at this time.

<u>Short-Term Remedial Work (Within 2 Years)</u>

2. Replace the steel cable guide rails in the west approach. The existing guide rails do not meet current provincial (MTO) standards, the wooden posts are deteriorated, and there is insufficient tension in the cables. This item is required by the MTO Roadside Safety Manual.
3. Inspect connection bolts in the bridge railings, tighten loose bolts and replace missing bolts.
4. Inspect all signs in the west approach, tighten loose bolts and replace missing bolts.

<u>Rehabilitation Work (Within 5 Years)</u>

5. Blast-clean and re-paint the structural steel. Currently the coatings are in generally poor condition and corrosion of the structural steel is advancing. With proper coatings in place, a structural steel bridge can last almost indefinitely; without proper coatings, the service life is limited and the cost of required repairs continues to grow. This item is required by the CHBDC for corrosion protection of superstructure steelwork. Concurrently with the re-coating contract, structural steel replacements and repairs should be carried out to the below-deck lateral bracing, the pivot hub steelwork, and other steel components of the bridge as required.
6. Replace deteriorated areas of the timber deck and timber curbs. Consider a complete re-decking of the bridge.
7. Replace the timber wearing surface.
8. Repair the undermining to the south-east corner of the timber cribbing below the south rest pier, below the waterline.

9. Replace the rotted and split timbers in the timber cribbing below the pivot piers and rest piers.
10. Perform concrete repairs and crack injections in the east pier. Consider re-facing the pier for a more complete rehabilitation that will reduce future repair contracts and avoid a "patchwork" appearance.
11. Re-face and re-surface the pivot pier.
12. Re-face and re-surface the rest piers.
13. Replace the damaged steel ladder on the east side of the north rest pier.
14. Replace the eroded material at the west abutment and north-west embankments and add erosion protection such stone rip-rap.
15. Replace the displaced slope protection stone rip-rap at the west abutment embankment.

Additional Engineering Studies / Investigations / Surveys

16. No additional engineering studies / investigations / surveys are recommended at this time.

Recommended Work - Class 'C' Cost Estimates					
Item No.	**Item Description**	**Unit**	**Quantity**	**Unit Price**	**Cost**
Immediate Remedial/Maintenance Work For Safety Reasons					
1	None recommended.	LS	1	$0	$0
Short-Term Remedial Work (Within 2 Years)					
2	Replace steel cable guiderail in west approach.	LS	1	$15,000	$15,000
3	Maintain bridge railing connections.*	LS	1	$2,500	$2,500
4	Maintain approach signage.*	LS	1	$5,000	$5,000
Rehabilitation Work (Within 5 Years)					
5	Clean and re-paint steelwork and perform steel replacements and repairs.	LS	1	$1,000,000	$1,000,000
6A	Replace deteriorated areas of timber deck and curbs.*	LS	1	$7,500	$7,500
6B	*Replace the timber deck.**	*LS*	*1*	*$50,000*	*$50,000*
7	Replace the timber wearing surface.*	LS	1	$25,000	$25,000
8	Repair the undermining to the south-east corner of the timber cribbing below the south rest pier, below the waterline.	LS	1	$10,000	$10,000
9	Replace the rotted and split timbers in the timber cribbing below the pivot piers and rest piers.	LS	1	$20,000	$20,000
10A	Perform concrete repairs and crack injections in the east pier	LS	1	See Fixed Bridge Cost Estimate	
10B	*Replace the east pier.*	*LS*	*1*	*See Fixed Bridge Cost Estimate*	

11	Re-face and re-surface the pivot pier.	LS	1	$20,000	$20,000
12	Re-face and re-surface the rest piers	LS	1	$250,000	$250,000
13	Replace the damaged steel ladder on the east side of the north rest pier.*	LS	1	$1,000	$1,000
14	Replace the eroded material at the west abutment and north-west embankments*	LS	1	$3,000	$3,000
15	Replace the displaced slope protection stone rip-rap at the west abutment embankment.*	LS	1	$10,000	$10,000
	Additional Engineering Studies / Investigations / Surveys				
16	None recommended.	LS	1	$0	$0
Notes: * Item possibly performed by PCA forces. - Items in italics are optional and not included in Total Estimated Cost.				**Sub-Total**	**$1,369,000**
				Contingency (25%)	**$342,250**
				Engineering (20%)	**$273,800**
				TOTAL ESTIMATED COST	**$1,985,050**

Table 13 – Estimated Structural Rehabilitation Costs for Swing Span

The Structural Financial Analysis Manual (SFAM) published by the MTO, provides assumed life spans for various bridge rehabilitation treatments, based on experience with bridges on high traffic volume highways. The SFAM indicates that the assumed life span of concrete re-facing treatments is from 10 to 20 years, depending on the level of exposure to chlorides. As the bridge is located on a low traffic volume road and is therefore assumed to be subject to low levels of chloride exposure, an assumed life span at the upper end of this range (i.e. 15-20 years) should be assumed. The SFAM does not provide assumed life spans for timber components.

10.1.2 Mechanical

Immediate Remedial/Maintenance Work for Safety Reasons

1. No immediate remedial/maintenance work is recommended at this time.

Short-Term Remedial Work (Within 2 Years)

2. Replace the balance wheel rail and adjust balance wheel clearance to the rail to remove live loading from the balance wheels.
3. Replace the end lift jacks with end lift machinery that meets the requirements of the CHBDC such as self locking screw jacks or a combination of jacks and separate end wedges. Ensure that end lift height is in accordance with CHBDC requirements.
4. Provide an energy absorbing stop at the full open position in accordance with CHBDC requirements.
5. Modify the existing hydraulic control system to include the following features:
   a. Pilot operated check valves at the cylinders to provide a holding function when the HPU is not energized.
   b. Counterbalance valves to provide a braking function.

c. A modified circuit that provides for two speed operation to allow the operator to bring the span into the full open and closed positions at reduced speeds and mitigate the potential for damaging impacts (required for automatic sequence control).

Rehabilitation Work (Within 5 Years)

6. Replace the obsolete traffic warning gates with standard commercial units.

Additional Engineering Studies / Investigations / Surveys

7. Although there were no obvious signs of problems with the center pivot assembly at the time of the inspection, consideration should be given to inspecting the wearing components of the center pivot in conjunction with any major rehabilitation work in light of the age of these components.

Recommended Maintenance

8. Clean and paint all machinery. Evaluate the section loss of all fasteners and anchor bolts and replace components as warranted.
9. Clean out the clogged balance wheel lubrication ports.
10. Replace damaged end castor anchor bolts and tighten loose anchor bolts.
11. Tighten both end lift base plates anchor bolts.
12. Adjust the east locking pin to restore functionality.
13. Replace the southeast end of travel stop energy absorbing pad.
14. Investigate the capacity and integrity of the southeast end of travel stop at the fully closed position and replace the anchor bolts as necessary.
15. Replace the northwest end stop anchor bolts.
16. Replace the abraded hydraulic hose located adjacent to the HPU in the operator's house.
17. Replace the flexible hydraulic hoses that connect the piping at the center pier to the hydraulic cylinders.
18. Adjust the blind end flexible hose connection from the HPU to the piping at the center pier to eliminate the severe bend radius.
19. Replace the west locking pin blind end hose.

Recommended Work - Class 'C' Cost Estimates					
Item No.	**Item Description**	**Unit**	**Quantity**	**Unit Price**	**Cost**
Immediate Remedial/Maintenance Work For Safety Reasons					
1	None recommended.	LS	1	$0	$0
Short-Term Remedial Work (Within 2 Years)					
2	Replace the balance wheel rail and adjust balance wheel clearance to the rail to remove live loading from the balance wheels.	LS	1	$20,000	$20,000
3	Replace the end lift jacks with end lift machinery that meets the requirements of the CHBDC such as self-locking screw jacks or a combination of jacks and separate end wedges. Ensure that end lift height is in accordance with CHBDC requirements.	LS	1	$30,500	$30,500
4	Provide an energy absorbing stop at the full open position in accordance with CHBDC requirements.	LS	1	$5,000	$5,000
5	Modify the existing hydraulic control system to meet CHBDC requirements.	LS	1	$15,000	$15,000
Rehabilitation Work (Within 5 Years)					
6	Replace the obsolete traffic warning gates with standard commercial units.	LS	1	*See electrical cost estimate*	
Additional Engineering Studies / Investigations / Surveys					
7	Jack the span to inspect the condition of the internal wearing components of the center pivot.	LS	1	$20,000	$20,000
Notes:				**Sub-Total**	**$90,500**
				Contingency (25%)	**$22,625**
				Engineering (20%)	**$18,100**
				TOTAL ESTIMATED COST	**$131,225**

Table 14 – Estimated Mechanical Rehabilitation Costs for Swing Span

10.1.3 Electrical

Immediate Remedial/Maintenance Work for Safety Reasons

1. No immediate remedial/maintenance work is recommended at this time.

Short-Term Remedial Work (Within 2 Years)

2. Provide a suitably sized standby power system to operate the bridge and allow the bridge to function seamlessly during the loss of main electric utility service.
3. Replace existing PLC controller with state of art modern PLC controller.
4. Provide a new control station to include status indication for all bridge operating equipment and include separate switches for the gates for independent operation.

New control station should also be provided with a means of disconnecting control station power to prevent unauthorized operation of the bridge.

5. Replace the end lift limit switch supports.
6. Provide limit switch for the east manually operated locking pin to prevent bridge operation when the pin is engaged.
7. Provide hand crank limit switch for the traffic control gates to prevent electrical operation of the gates when the hand crank handle is inserted.
8. Provide door limit switches to prevent traffic control gate operation when enclosure door is removed.
9. Provide fender navigation lights for channel marking as per coast guard requirement.
10. Provide emergency lighting and exit signs for bridge control building.
11. Install a means of operating and emergency stopping all hydraulic drives locally for the safety of maintenance personnel.

Rehabilitation Work (Within 5 Years)

12. Replace the obsolete traffic warning gates with standard commercial units.

Additional Engineering Studies / Investigations / Surveys

13. Investigate the full load output of the HPU pump motor during the bridge operation, the HPU appears to be undersized for the prevailing duty.

Recommended Maintenance

14. Remove the nuts between the cover plate and the junction box for the swing span hydraulic pump motor to provide complete seal of the junction box.
15. Replace existing enclosure cover for the swing span hydraulic pump motor starter with a cover that is compatible with the current starter so reset button on the cover can be utilized.
16. Remove all spare parts and wires in the relay/contactor panel and locate them in a common storage area for spare parts.
17. Replace the broken LED light for the solid state contactor in the relay/contactor panel.
18. Properly terminate all unused or spare wires in all junction boxes and enclosures.
19. Provide label for the push button located at the bottom right corner of the control station.
20. Clean and remove corrosion on all Teck cable, conduit and hydraulic host fittings.
21. Clean and remove corrosion on all limit switch roller arms, replace as required.
22. Secure the mounting plate for the locking pin.
23. Clean and remove debris for the bridge nearly closed and fully closed limit switches.
24. Repaint the west approach Stop Bar.
25. Clean and remove corrosion and repaint all traffic gate enclosures.

Recommended Work - Class 'C' Cost Estimates					
Item No.	**Item Description**	**Unit**	**Quantity**	**Unit Price**	**Cost**
	Immediate Remedial/Maintenance Work For Safety Reasons				
1	None recommended.	LS	1	$0	$0
	Short-Term Remedial Work (Within 2 Years)				
2	Provide a suitably sized standby power system.	LS	1	$50,000	$50,000
3	Replace existing PLC controller with state of art modern PLC controller.	LS	1	$15,000	$15,000
4	Provide a new control station.	LS	1	$10,000	$10,000
5	Replace the end lift limit switch supports.	LS	1	$500	$500
6	Provide limit switch for the east manually operated locking pin.	LS	1	$2,000	$2,000
7	Provide hand crank limit switch for the gates.	LS	1	$1,000	$1,000
8	Provide door limit switches to prevent gate operation when enclosure door is removed.	LS	1	$1,000	$1,000
9	Provide fender navigation lights for channel marking as per coast guard requirement.	LS	1	$18,000	$18,000
10	Provide emergency lighting and exit signs for bridge control building.	LS	1	$5,000	$5,000
11	Install a means of operating and emergency stopping all hydraulic drives.	LS	1	$5,000	$5,000
	Rehabilitation Work (Within 5 Years)				
12	Replace existing obsolete traffic gates with standard commercial traffic gates.	LS	1	$25,000	$25,000
	Additional Engineering Studies / Investigations / Surveys				
13	Investigate the full load output of the HPU pump motor during the bridge operation.	LS	1	$5,000	$5,000
Notes:				**Sub-Total**	**$137,500**
				Contingency (25%)	**$34,375**
				Engineering (20%)	**$27,500**
				TOTAL ESTIMATED COST	**$199,375**

Table 15 – Estimated Electrical Rehabilitation Costs for Swing Span

10.2 Fixed Bridge - Structural

Immediate Remedial/Maintenance Work for Safety Reasons

1. Install temporary strengthening measures for the most easterly bottom chord truss members, due to the severely corroded bottom chord I-bars at the east truss bearings (already completed).

Short-Term Remedial Work (Within 2 Years)

2. Install new bottom chord members in the most easterly truss panels, at the locations of the above temporary strengthening measures. Consider replacing all bottom chord members with stronger members, if the bridge is to be left in service for an extended period of time. This item is required by the CHBDC, for strength and durability of the bridge.
3. Replace the steel cable guiderail in the east approach. This item is required by the MTO Roadside Safety Manual.
4. Replace the eroded material at the northeast embankment and add slope protection such as stone rip-rap or concrete.
5. Replace the displaced erosion protection rip-rap in front of the east abutment. The geotechnical report recommends at least 1 metre of rip-rap for a durable layer of protection to withstand water and ice flows.
6. Replace the damaged signs and secure loose signs in the east approach.
7. Asphalt-patch the depressions in the east approach pavement to prevent ponding, and rout and seal cracks.

Rehabilitation Work (Within 5 Years)

8. If the current bridge is to be maintained in use for the foreseeable future, blast-clean and re-coat the structural steel. As with the swing span, the paint coatings are currently in poor condition and need to be replaced if the bridge is to be left in service. Consider removing and replacing the entire superstructure rather than re-painting it in-situ. This item is required by the CHBDC for corrosion protection of superstructure steelwork.
9. Repair/replace the roller bearings at the west end of the bridge.
10. Replace deteriorated areas of the timber deck and curbs. Consider re-decking the bridge if it is to be left in service for an extended period.
11. Replace the timber running boards.
12. Repair or replace impact-damaged steel members in the end portal frames. This item is required by the CHBDC, for strength and durability of the bridge.
13. Perform concrete repairs and crack injections in the east pier. If replacement of the current bridge is being considered, then consideration should also be given to the complete replacement of the pier, as the compressive strength of the existing concrete is relatively low.
14. Replace the north and south pins and housing at L7. This item is required by the CHBDC, for strength and durability of the bridge.
15. If the current bridge is to be maintained in use for the foreseeable future, underpin the east abutment with micropiles down to bedrock, and re-face the exposed

surfaces of the abutment with concrete. These recommendations are based on the results of the geotechnical investigation at the east abutment (see Section 3 of this report).

If the bridge is to be replaced, remove and replace the abutment with a new concrete abutment founded on micropiles socketed into bedrock.

Additional Engineering Studies / Investigations / Surveys

16. If it is contemplated to raise the current load limit of 3 tonnes, with the bridge in its current condition, perform additional inspection and testing of the stringers and stringer bearings to verify their tensile capacity (per Sections 7.2 and 7.3 of this report).

Recommended Maintenance

17. Flush dirt and debris from bearing seats and deck surface (on-going).

Recommended Work - Class 'C' Cost Estimates					
Item No.	**Item Description**	**Unit**	**Quantity**	**Unit Price**	**Cost**
	Immediate Remedial/Maintenance Work For Safety Reasons				
1	Temporary strengthening for bottom chords at east bearings (completed) *	LS	1	$9,500	$9,500
	Short-Term Remedial Work (Within 2 Years)				
2A	Replace truss bottom chords at east bearings.	LS	1	$20,000	$20,000
2B	*Replace all truss bottom chords.*	*LS*	*1*	*$140,000*	*$140,000*
3	Replace steel cable guiderail in east approach.	LS	1	$7,000	$7,000
4	Northeast embankment slope protection.*	LS	1	$3,500	$3,500
5	Rip-rap in front of east abutment.*	LS	1	$17,000	$17,000
6	Replace damaged signage in east approach.*	LS	1	$5,000	$5,000
7	Patch asphalt in east approach and seal cracks.*	LS	1	$2,500	$2,500
	Rehabilitation Work (Within 5 Years)				
8A	Clean and re-paint structural steel and perform minor steel repairs.	LS	1	$750,000	$750,000
8B	*Remove and replace the superstructure.*	*LS*	*1*	*$1,500,000*	*$1,500,000*
9	Repair/replace roller bearings.*	LS	1	$17,000	$17,000
10A	Replace deteriorated areas of timber deck and curbs.*	LS	1	$7,500	$7,500
10B	*Replace timber deck.**	*LS*	*1*	*$35,000*	*$35,000*
11	Replace timber running boards.*	LS	1	$9,000	$9,000
12	Repair/replace damaged portal frame members.	LS	1	$25,000	$25,000
13A	Concrete repairs to east pier.	LS	1	$30,000	$30,000
13B	*Replace the east pier (including dewatering).*	*LS*	*1*	*$300,000*	*$300,000*

14	Replace the north and south pins at L7.	LS	1	$17,000	$17,000
15A	Underpin and re-face the east abutment.	LS	1	$250,000	$250,000
15B	*Replace the east abutment.*	*LS*	*1*	*$375,000*	*$375,000*
Additional Engineering Studies / Investigations / Surveys					
16	Additional investigation of stringers if bridge load limit to be raised.	LS	1	$12,000	$12,000
Notes: * Items possibly performed by PCA forces. - Items in italics are optional and not included in Total Estimated Cost				**Sub-Total**	**$1,182,000**
				Contingency (25%)	**$295,500**
				Engineering (20%)	**$236,400**
				TOTAL ESTIMATED COST	**$1,713,900**

Table 16 – Estimated Structural Rehabilitation Costs for Fixed Span.

The Structural Steel Coating Manual (SSCM) published by the MTO (revised in April 2004) indicates that the estimated service life of a new coating system (following complete removal of the existing coating) is from 20 to 25 years.

The Structural Financial Analysis Manual (SFAM) published by the MTO, provides assumed life spans for various bridge rehabilitation treatments, based on experience with bridges on high traffic volume highways. The SFAM indicates that the assumed life span of concrete re-facing treatments is from 10 to 20 years, depending on the level of exposure to chlorides. As the bridge is located on a low traffic volume road and is therefore assumed to be subject to low levels of chloride exposure, an assumed life span at the upper end of this range (i.e. 15-20 years) should be assumed. The SFAM does not provide assumed life spans for timber components.

10.3 Cost Estimate Summary

The following table provides a summary of estimated costs for the recommended structural, mechanical and electrical rehabilitation work for both spans combined.

Recommended Work - Class 'C' Cost Estimates					
Item No.	**Item Description**	**Unit**	**Quantity**	**Unit Price**	**Cost**
Immediate Remedial/Maintenance Work For Safety Reasons					
---	Structural, mechanical and electrical *	LS	1	$9,500	$9,500*
Short-Term Remedial Work (Within 2 Years)					
---	Structural, mechanical and electrical	LS	1	$255,500	$255,500
Rehabilitation Work (Within 5 Years)					
---	Structural, mechanical and electrical	LS	1	$2,477,000	$2,477,000
Additional Engineering Studies / Investigations / Surveys					
---	Structural, mechanical and electrical	LS	1	$37,000	$37,000
Notes: * Work is already completed.				**Sub-Total**	**$2,779,000**
				Contingency (25%)	**$694,750**
				Engineering (20%)	**$555,800**
				TOTAL ESTIMATED COST	**$4,029,550**

Table 17 – Estimated Rehabilitation Costs for Both Spans Combined.

11. CONCLUSIONS

11.1 Structural Conclusions

11.1.1 Swing Span

Based on the inspections, investigations and assessments carried out, the following was established regarding the swing span.

The swing span structure is in generally fair to good condition except for the structural steel coatings, the below-deck lateral bracing and the sides of the concrete pivot pier and rest piers. The steel coatings in particular should be a priority for renewal to preserve the steelwork and reduce future repair costs. Testing of paint samples from the span indicated high levels of lead.

In its current condition, the swing span could be triple load-posted to 19 tonnes for single-unit vehicles, 34 tonnes for two-unit vehicles and 43 tonnes for vehicle trains. However, the fixed span load limit is lower and so governs the load limit for the crossing, as summarized below.

The overall **Structural Condition Rating is 2 (Inadequate)**, based on the allowable load postings according to the results of the structural evaluation. The overall **Functional Rating is also 2 (Inadequate)**, for the same reason. These ratings criteria are clearly identified in the 2010 BIM.

Recommended short-term remedial work (within two years) on the span includes: Replacement of the steel cable guide rails in the west approach with MTO-approved guiderail; Maintenance of the bridge railing connections; and maintenance of approach signage.

Recommended rehabilitation work (within 5 years) on the span includes: Cleaning and re-painting of the steelwork and minor steel repairs; and replacement of deteriorated areas of timber curbs and deck.

The estimated cost of the recommended structural work is about $2.0M, including contingency and engineering costs, but excluding taxes.

11.1.2 Fixed Span

Based on the inspections, investigations and assessments carried out, the following was established regarding the fixed span.

The fixed span structure is in generally fair to good condition, with the exception of the steel coatings, truss bottom chords and the east abutment. Extreme deterioration of the bottom chords was observed at the east bearings and has been addressed through installation of cables at these locations. If it is decided to maintain the current bridge in use rather than replace it, the truss bottom chords and steel coatings should be a priority for renewal to preserve the safety of the structure and reduce future repair costs. Testing of paint samples from the swing span indicated high lead levels.

The structural review and evaluation of the span concluded that the stringers are sharing tensile load with the truss bottom chords. This explains why the span is able to support higher loads than would otherwise be possible based on the very slender and deteriorated bottom chord bars. The continuity of the stringers and their connections to the east abutment and east pier are likely a contributor to the distress and movements observed at the east abutment.

In its current condition, with the bottom chords at the east end replaced, the fixed span could be triple load-posted to 12 tonnes for single-unit vehicles, 18 tonnes for two-unit vehicles and 19 tonnes for vehicle trains. However, if PCA decides to raise the load limit for the crossing from the current 3 tonnes, destructive testing of the stringer connections to the east abutment should be performed, prior to changing the load posting.

The overall **Structural Condition Rating is 2 (Inadequate)**, based on the allowable load posting according to the results of the structural evaluation. The overall **Functional Rating is also 2 (Inadequate)**, for the same reason. These ratings criteria are clearly identified in the 2010 BIM.

The recommended immediate work for safety reasons was the strengthening of the truss bottom chords at the east truss bearings, and this work has already been completed by PCA forces.

Recommended short-term remedial work (within two years) on the span includes: Replacement of the truss bottom chords at the east bearings; Replacement of the steel cable guiderail in the east approach; Installation of slope protection at the northeast embankment; Installation of stone rip-rap in front of the east abutment; Maintenance of the east approach signage; and patching of approach asphalt and sealing asphalt cracks.

Recommended rehabilitation work (within 5 years) on the span includes: Cleaning and re-painting of the steelwork and minor steel repairs; Repair/replacement of the roller bearings; Replacement of deteriorated areas of timber curbs and decking; Replacement of the timber running boards on the deck; Repair of the damaged portal frame members; Concrete repairs to the east pier; Replacement of the steel connection pins at L7; and underpinning and re-facing of the east abutment.

The estimated cost of the recommended structural work is about $1.7M, including contingency and engineering costs, but excluding taxes. The estimated cost of a new fixed span and new east abutment and east pier is about $3.3M, including contingency and engineering.

11.2 Mechanical Conclusions

The condition of the mechanical machinery systems ranges from good to poor. In addition to deterioration due to aging and corrosion, several machinery systems do not meet current design requirements. Significant repairs and modifications are recommended to address existing condition and design deficiencies.

11.3 Electrical Conclusions

The existing electrical system provides power, control and safety logic for the bridge hydraulic system as well as providing general power and lighting for the bridge control building. The electrical control system and hydraulic system was replaced in the 1991/1992 season.

The utility service is derived from an overhead medium voltage line and a single pole mounted transformers that provides 120/240V, single phase service to the bridge, with no electrical standby or backup service. The installation is generally in serviceable condition with minor to moderate signs of corrosion for some exterior equipment and their installation. The existing PLC controller should be replaced with a state of art modern PLC controller to eliminate electrical system obsolescence.

In addition, consideration should be given to replacing the existing custom traffic gates with commercially available roadway traffic gates and installing a standby generator for emergency operation of the bridge.

12. CLOSURE

We trust that this report contains sufficient information for your present purposes. If you have any questions regarding this report, please contact us.

Yours truly,

DELCAN CORPORATION

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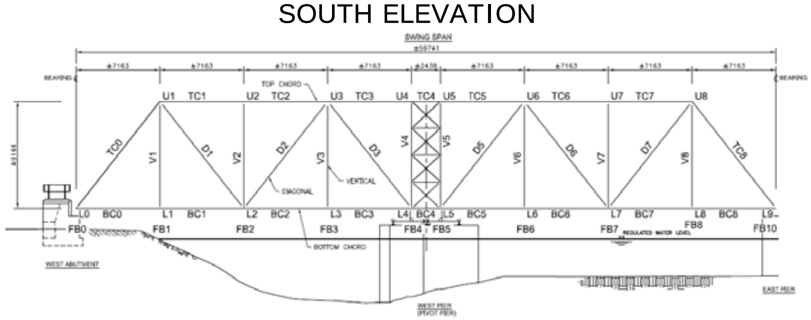
APPENDIX A

INSPECTION FORMS & MCR/PCR FORMS

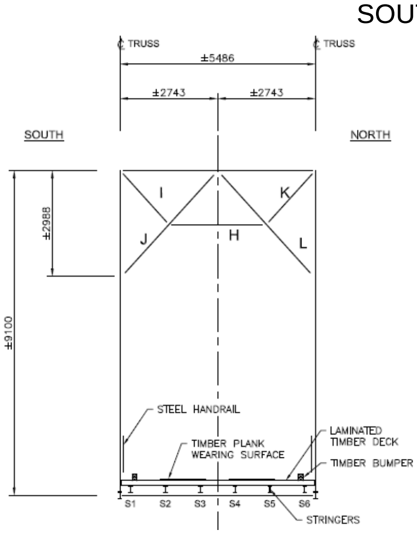
INSPECTION FORM

NAME:	Hamlet Bridge (Bridge 57) – Swing Span
LOCATION:	Canning Road, Hamlet, Ontario
YEAR CONSTRUCTED:	Circa 1905-1922 *

SOUTH ELEVATION



SOUTH ELEVATION



CROSS-SECTION

Notes:

1. Equal-arm through-truss swing bridge.
2. Timber deck and timber plank wearing surface.
3. Central swing pier and rest piers are comprised of timber cribbing with concrete blocks and cast-in-place concrete caps.

* Superstructure built circa 1905, substructures built circa 1915-1922

INSPECTION FORM

NAME:	Hamlet Bridge (Bridge 57) – Swing Span
LOCATION:	Canning Road, Hamlet, Ontario
YEAR CONSTRUCTED:	Circa 1922
TYPE OF INSPECTION:	Comprehensive Detailed Inspection
Original Design:	Department of Railways and Canals, 1921
Drawings Available:	Yes
Previous Inspection Report Date:	None
Author:	N/A
Current Inspection Date:	September 28 and 29, 2011
Inspectors:	Patrick Mergel, P.Eng., ing.; Ben MacMaster, P.Eng.; Peter Harvey, EIT.
Temperature:	15°C-21°C (28th); 13°C-18°C (29th);
Weather:	Rain a.m., sunny p.m. (28th); Mainly cloudy, late thunderstorms (29th)
Equipment:	Dive boat supplied by Lower Lakes Marine Pontoon boat supplied by Loon Wing Lift Services Bucket truck supplied by Rostance Electric
Previous Condition Rating:	None
Previous Functional Rating:	None
Current Structural Condition Rating:	2
Current Functional Rating:	2

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Waterway (P)	No significant defects noted.	6	D	
Foundations (P)	No signs of foundation problems noted. See notes below concerning pier substructures below the waterline.	6	D	
Abutments (P)	West abutment wall is in good condition. No significant defects noted. West abutment ballast wall is in good condition. No significant defects other than a single vertical crack and a rust stain were noted. West bearing seat is in good condition. Small areas of light honeycombing and medium scaling on the north-west wingwall, plus a small spall at the top at the west end.	5	D/M	S19, S20
Girders (Trusses) (P)	Condition rating based on MCR of numerous lower connections. Extensive areas of coating failure and light corrosion on majority of members. Several gusset plates at upper truss connections are bent due to rust jacking. Top layers of steel have completely delaminated on a south truss diagonal bracing member between panel points 4 and 5.	3	B	S21-S25, S34-S36

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Floor System (P)	Condition rating based on numerous floor beams and stringers MCR. Floor beams: Extensive areas of coating failure and light to very severe corrosion on majority of members. Severe localized section loss of many members, including of web at connections to stringers, and on bottom flange at many locations. Stringers: Extensive areas of coating failure and light to severe corrosion on majority of members. Localized areas of 20% to 30% section losses at base of web are typical. Gap between bottom flange and supporting bracket plus severe rust jacking of angles at several floor beam connections.	3	B	S26-S30
Coatings (P, S)	The coatings are in generally very poor condition throughout, with extensive areas of cracked and flaking noted. Apparent red lead primer observed.	1	B	S21-S39
Deck (P)	Timber beams at ends of deck are severely rotted and split. Checking, rot and splitting of numerous deck members. Fractured deck member midway between FB0 and FB1 and S0-3 and S0-4.	4	B	S40-S43
Wearing Surface (P)	Numerous wearing surface boards rotted, particularly at the ends of the bridge. Inside edges of the boards either side of the central longitudinal section of deck sound hollow and have light abrasion along entire length of the deck. Several other boards have rotten sections. Plywood shim beneath north wearing surface boards at east end of deck is rotten.	3	B	S40-S42
Pivot Structural Steel (P)	Extensive coating failure and numerous areas of severe localized section loss. Central girder previously strengthened to repair cracks at bottom of web. PCA representative reported cracks were welded and vertical stiffeners added approximately 10 years ago. Severe section loss of bottom flange and rivet heads at connections with bracing members, including perforations in gusset plates. Ends of bracing severely corroded at connections. Localized section of west bottom flange of girder beneath FB4 has 500 mm long section of severe section loss. Localized section of north-west bottom flange of hub at pivot has 200 mm long section of very severe section loss. Bottom flange of member connecting 2 hub members at pivot has 150 mm x 50 mm perforation. Top and bottom gusset plates connecting hub member and diagonal bracing at pivot have 50% localized section loss.	3	B	S37-S39

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Piers (P)	Centre Swing Pier: Several very large areas of medium and severe scaling, area of cracked grout beneath east side of rail and several wide cracks on the pier top. Several large areas of severe and very severe scaling on the pier sides, and very severe erosion along the length of the pier at the waterline. North Rest Pier: Numerous narrow to wide transverse cracks, large spalls, and large areas of severe scaling are typical in the top of the concrete cap. Several depressed areas with ponding water along the longitudinal centreline of the pier. North-east corner of second section of slab from south has settled by 25 mm. 30 mm depression measured at centre of construction joint between third and fourth sections from south. Wide map cracks over entire surface of wall on top of pier at north end. Numerous large areas of severe and very severe scaling and spalled concrete in the sides of the concrete cap and concrete blocks below. Numerous wide vertical cracks in the side of the concrete cap, and deep spalls/disintegration at the interface of the concrete cap and concrete blocks. Several large, deep spalls in the concrete blocks have exposed the steel lifting hooks. Some minor rotting was noted on the corners and ends of several underwater timber crib members, and some gaps between the ends of adjacent timbers at the riverbed. Several entire timbers are rotten or have split longitudinally. The top timber at the north end of the east side is loose. The ends of numerous cross ties are rotten, one up to a depth of 430 mm. The steel ladder on the east side of the pier is severely bent in the downstream direction. South Rest Pier: Several narrow to wide transverse cracks, large areas of medium to severe scaling and several areas of ponding water along longitudinal centreline of pier top. Sagging of 25 mm at centre of second section from south. Severe scaling and map cracks over entire surface of wall on top of pier at south end. Large cracks between steel nosing plates and a small tree growing at south end. Numerous large areas of severe and very severe scaling and spalled concrete in the sides of the concrete cap and blocks. Numerous wide vertical cracks in the side of the concrete cap, and deep spalls/disintegration at the interface of the concrete cap and concrete blocks. Several large and deep spalls in the concrete blocks have exposed the steel lifting hooks. There is a large void beneath the steel nosing plates at the south end.	2	B	S6-S18

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
	The ends of the crossties at the south end of the west side are generally rotten. There are several 250 mm x 250 mm voids in the east side of the cribbing where the ends of the longitudinal timbers have rotted away. There is a 330 mm gap between the cribbing and the riverbed at the south-east corner of the pier that tapers to 0 mm over a length of approximately 2.5 m. <u>East Pier:</u> The grout/concrete filled bags below the waterline were found to be in generally good condition with no significant undermining noted. Numerous transverse cracks and areas of map cracking in the inclined section of the concrete cap, with a large delaminated area at the base at the south-west corner and efflorescence at the bottom edge on the west side. Several areas of severe scaling and disintegration on upper vertical shaft, plus horizontal cracks with efflorescence at ends of upper shaft. Areas of severe disintegration and spalling at interface of inclined section and lower pier shaft. The east bearing seat on the east pier is in good condition. Accumulation of dirt and debris typically noted.			
Curbs (S)	Light abrasion along the length of curbs, and minor splits and checks typical. Member at west end of north curb is loose and splitting longitudinally. The member between FB3 and FB4 on the south curb has a 6 mm wide longitudinal split along its length. The north and south members at the east end are not tapered.	5	B	S43, S46
Bottom Chord Bracing (S)	Extensive coating failure and light to severe corrosion on majority of members. Three small perforations in 6N-7S. 200 mm long perforation in 5S-6N. 75 mm x 50 mm perforation and larger area of severe section loss in 2N-3S.	2	B	S31-S33
Upper Sway Bracing (S)	Extensive coating failure and light corrosion on majority of members. North diagonal members are bent in west portal frame. Water is ponding and moss is growing in bottom of many lateral members. Many diagonal bracing members bent or sagging.	4	B	S21, S24
Deck Joints (S)	The joints at west and east end of bridge are open joints, allowing dirt, debris and rain/snow to fall onto the bearing seats.	5	D	
Approaches (S)	Areas of light ravelling at centreline and south side of west approach wearing surface, and light abrasion on end dam. End dam is sloped to allow smooth passage onto bridge but creates uneven ride for vehicles.	5	D	S48

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Railings (S)	The bridge railings do not meet current CHBDC crash tested requirements. Several loose and missing bolts in connections to posts on both north and south railings. Numerous locations of impact damage to lattice, end balusters, and to top and lower rails. Coating has typically failed on at least 30% of lattice and at least 50% of top and bottom rails, with light to medium corrosion developing.	3	A	S44-S47
Approach Guiderails (S)	The approach guiderails do not meet current provincial standards. The steel cables on both the north and south sides of the west approach have tension loss. The west end of the steel cable on the south side of the approach is attached to a road sign post. The first ten posts at the east end on the south side are severely rotted. The steel tube railing on the north side of the approach has slight impact damage and small areas of coating failure. The steel tube railing posts on the south side only have 2 of 4 anchor bolts installed.	1	A	S48-S50
Embankments (S)	Approximately 10% erosion at the end of and adjacent to the north-west wingwall due to water runoff from the roadway, with a large tree growing near the wingwall. Small trees growing in front of west abutment wall. Some erosion of embankment material in front of west abutment wall.	4	B	S19
Slope Protection (A)	Some slope protection stones at the west abutment embankment have been displaced.	5	B	S19
Utilities (A)	The light at the south-west corner of the truss is broken.	N/A	D	
Signs (A)	The street name and traffic light sign posts in the west approach are not plumb. Bottom bolt is missing from the “slippery road” sign at the west end of the north truss. The “hazard close to edge of road” sign at the west end of the south truss is loose and has some impact damage. The “stop here on red signal” sign on the west approach is loose.	N/A	A/M	S51

RECOMMENDATIONS

1. Replace the steel cable guide rails in the west approach (1 year).
2. Replace the missing bolts in the bridge railings (1 year).
3. Secure loose signs (1 year).
4. Repair deteriorated structural steel members in the floor system, trusses and pivot hub. Bottom lateral bracing are most deteriorated members and highest priority (3 years)
5. Blast-clean and re-coat the structural steel (3 years).
6. Replace deteriorated areas of timber deck and timber curbs (3 years).

7. Replace the timber wearing surface (3 years).
8. Repair the undermining to the south-east corner of the timber cribbing below the south rest pier, below the waterline (3 years).
9. Replace the rotted and split timbers in the timber cribbing below the pivot piers and rest piers (3 years).
10. Perform concrete repairs and crack injections in the east pier (3 years). Consider re-facing the pier for a more complete rehabilitation that will reduce future repair contracts and avoid a "patchwork" appearance.
11. Re-face and re-surface the pivot pier (3 years).
12. Re-face and re-surface the rest piers, or completely replace them (3 years).
13. Replace the damaged steel ladder on the east side of the north rest pier (3 years).
14. Replace the eroded material at the west abutment and north-west embankments and add erosion protection such stone rip-rap (3 years).
15. Replace the displaced slope protection stone rip-rap at the west abutment embankment (3 years).
16. Flush dirt and debris from the bearing seats (on-going).

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Top Chord						
Truss	TC0	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	Coating over 90% of inside surfaces is flaking.
Truss	TC1	None	None	4	5	Light corrosion over 20% of top plate, 20% on inside C-channel.
Coating		None	None	2	2	20% of coating on top plate gone, 20% on inside C-channel.
Truss	TC2	None	None	5	5	Light corrosion over 20% of top plate.
Coating		None	None	3	3	20% of coating on top plate gone.
Truss	TC3	None	None	5	5	Light corrosion over 20% of top plate.
Coating		None	None	3	3	20% of coating on top plate gone.
Truss	TC4	None	None	4	5	Light corrosion over 20% of top plate, 20% on inside C-channel. Ponding on top plate at west end.
Coating		None	None	2	2	20% of coating on top plate gone, 20% on inside C-channel.
Truss	TC5	None	None	4	5	Light corrosion over 20% of top plate, 30% on inside C-channel.
Coating		None	None	2	2	20% of coating on top plate gone, 30% on inside C-channel.
Truss	TC6	None	None	4	5	Light corrosion over 20% of top plate, 30% on inside C-channel.
Coating		None	None	2	2	20% of coating on top plate gone, 30% on inside C-channel.
Truss	TC7	None	None	4	5	Light corrosion over 20% of top plate, 30% on inside C-channel.
Coating		None	None	2	2	20% of coating on top plate gone, 30% on inside C-channel.
Truss	TC8	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating has flaked off.
Bottom Chord						
Truss	BC0	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC1	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	4	4	Coating over 2% of member is flaking.
Truss	BC2	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC3	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC4	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC5	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC6	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC7	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	4	4	Coating over 5% of member is flaking.
Truss	BC8	None	None	5	5	Light corrosion over 5% of member.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	4	4	Coating over 5% of member is flaking.
Diagonals						
Truss	D1	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	D2	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	D3	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	D5	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	1	Coating is flaking off in large sheets on inside faces of c-channels.
Truss	D6	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	D7	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Verticals						
Truss	V1	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V2	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	V3	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	V4	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	V5	None	None	4	5	Light corrosion over 15% of member.
Coating		None	None	3	3	15% of coating has flaked off.
Truss	V6	None	None	4	5	Light corrosion over 20% of member. Light pitting on inside flange at U6.
Coating		None	None	2	2	20% of coating has flaked off.
Truss	V7	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V8	None	None	4	5	Light corrosion over 80% of top half of member.
Coating		None	None	1	1	80% of coating of top half of member has flaked off.
Central Bay Bracing						
Diagonal Bracing	4-5	None	None	4	5	Light corrosion over 10% of members.
Coating		None	None	4	4	10% of coating has flaked off.
Lower Connections						
Truss	L0	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Truss	L1	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Truss	L2	None	None	3	5	Light to medium corrosion over 100% of top horizontal plate and 80% of vertical gusset plates.
Coating		None	None	1	1	Coating has gone over 100% of top horizontal plate and 80% of vertical plates.
Truss	L3	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Truss	L4	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Truss	L5	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Truss	L6	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Truss	L7	None	None	3	5	Light to medium corrosion over 100% of top and bottom horizontal plates.
Coating		None	None	1	1	Coating has gone over 100% of top and bottom horizontal plates.
Truss	L8	None	None	3	5	Light corrosion 50% of top horizontal plate.
Coating		None	None	1	1	Coating has gone over 50% of top horizontal plate.
Truss	L9	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate.
Upper Connections						
Truss	U1	None	None	4	5	Rust jacking and light corrosion over 70% of top plate on D0.
Coating		None	None	1	1	30% of coating gone on top plate, 80% on top plate of D0.
Truss	U2	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U3	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 90% of top plate due to coating failure.
Coating		None	None	1	1	90% of coating gone on top plate.
Truss	U4	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 70% of top plate due to coating failure.
Coating		None	None	1	1	70% of coating gone on top plate.
Truss	U5	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 50% of top plate due to coating failure.
Coating		None	None	1	1	50% of coating gone on top plate.
Truss	U6	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U7	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 30% of top plate due to coating failure.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U8	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Top Chord						
Truss	TC0	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating gone.
Truss	TC1	None	None	4	5	Light corrosion over 20% of top plate, 10% on inside C-channel. Moss growing on bottom flange.
Coating		None	None	2	2	20% of coating on top plate gone, 10% on inside C-channel.
Truss	TC2	None	None	4	5	Light corrosion over 20% of top plate.
Coating		None	None	3	3	20% of coating on top plate gone.
Truss	TC3	None	None	4	5	Light corrosion over 20% of top plate.
Coating		None	None	3	3	20% of coating on top plate gone.
Truss	TC4	None	None	4	5	Light corrosion over 20% of top plate, 30% on inside C-channel. Moss growing on bottom flange at west end. Ponding on top plate.
Coating		None	None	2	2	20% of coating on top plate gone, 30% on inside C-channel.
Truss	TC5	None	None	4	5	Light corrosion over 20% of top plate, 10% on inside C-channel. Moss growing on bottom flange at west end.
Coating		None	None	2	2	20% of coating on top plate gone, 10% on inside C-channel.
Truss	TC6	None	None	4	5	Light corrosion over 20% of top plate, 30% on inside C-channel.
Coating		None	None	2	2	20% of coating on top plate gone, 30% on inside C-channel.
Truss	TC7	None	None	4	5	Light corrosion over 20% of top plate.
Coating		None	None	3	3	20% of coating on top plate gone.
Truss	TC8	None	None	4	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating gone.
Bottom Chord						
Truss	BC0	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC1	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC2	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC3	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC4	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC5	None	None	4	5	Light corrosion over 40% of member.
Coating		None	None	1	1	40% of coating flaked off.
Truss	BC6	None	None	4	5	Light corrosion over 20% of member. Rust jacking of plates at splice.
Coating		None	None	1	1	20% of coating flaked off. Further 30% of coating is cracked.
Truss	BC7	None	None	4	5	Light corrosion over 20% of member.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	1	1	20% of coating flaked off. Further 30% of coating is cracked.
Truss	BC8	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	20% of coating flaked off. Further 20% of coating is cracked.
Diagonals						
Truss	D1	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	2	2	Coating is peeling off in large sheets.
Truss	D2	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating gone.
Truss	D3	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating gone.
Truss	D5	None	None	4	5	Light corrosion over 30% of upper half of member.
Coating		None	None	1	1	30% of coating of upper half of member has flaked off.
Truss	D6	None	None	4	5	Light corrosion over 50% of upper half of member.
Coating		None	None	1	1	50% of coating of upper half of member has flaked off.
Truss	D7	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating gone.
Verticals						
Truss	V1	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	3	3	10% of coating has flaked off.
Truss	V2	None	None	4	5	Light corrosion over 50% of inside flange.
Coating		None	None	1	1	50% of inside coating has flaked off.
Truss	V3	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V4	None	None	4	5	Light corrosion over 30% of member. Counterweights at base of member will retain moisture and accelerate deterioration.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V5	None	None	4	5	Light corrosion over 30% of member, and over 90% at bottom 2m.
Coating		None	None	1	1	30% of coating has flaked off, and over 90% at bottom 2m.
Truss	V6	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V7	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Truss	V8	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating has flaked off.
Central Bay Bracing						
Diagonal Bracing	4-5	None	None	4	4	Bottom diagonal bracing-top layer of steel has delaminated from angle.
Coating		None	None	2	2	20% of coating has flaked off.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Lower Connections						
Truss	L0	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L1	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L2	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L3	None	None	4	5	Light corrosion over 10% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked/cracked.
Truss	L4	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L5	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L6	None	None	3	5	Light to medium corrosion over majority of top horizontal plate.
Coating		None	None	1	1	Coating has gone over majority of top horizontal plate. Coating has cracked on underside of bottom plate.
Truss	L7	None	None	3	5	Medium corrosion on bottom gusset plate.
Coating		None	None	1	1	100% coating failure on bottom gusset plate.
Truss	L8	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Truss	L9	None	None	4	5	Light corrosion over 50% of horizontal plate.
Coating		None	None	1	1	50% of horizontal plate has flaked off.
Upper Connections						
Truss	U1	None	None	4	4	Upper plate on D0 is bent due to rust jacking. Light corrosion over 20% of top plate.
Coating		None	None	1	1	50% of coating flaked off.
Truss	U2	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U3	None	None	4	5	Light corrosion over 70% of top plate. Bird nest on bottom flange of top chord.
Coating		None	None	1	1	70% of coating on top plate flaked off.
Truss	U4	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U5	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U6	None	None	4	5	Light corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U7	None	None	4	4	Upper plate bent due to rust jacking. Bird nest on lower gusset plate. Light

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						corrosion over 30% of top plate due to coating failure.
Coating		None	None	1	1	30% of coating gone on top plate.
Truss	U8	None	None	4	4	Upper plate bent due to rust jacking. Light corrosion over 30% of plates typical due to coating failure, particularly on bottom flange of C-channels.
Coating		None	None	1	1	30% of coating flaked off.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Primary Components

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Floorbeams						
Floorbeam	FB0	None	None	3	4	Medium corrosion on 50% of underside of bottom flange. Severe corrosion and deep pitting on top flange and web at connection to end stringers. Localized section of east web at connection to S0-4 has 73% section loss (ultrasonic test). Localized section of east web at connection to S0-2 has 20% section loss (ultrasonic test). Localized section of east web at connection to S0-3 has 23% section loss (ultrasonic test).
Coating		None	None	3	3	50% of coating has failed on underside of bottom flange.
Floorbeam	FB1	None	None	3	4	2mm pitting in web at connection to stringers typical. Medium corrosion on underside of top flange at connection to stringers. Localized section of west web at connection to S0-2 has 31% section loss (ultrasonic test). Localized section of east web at connection to S0-2 has 49% section loss (ultrasonic test).
Coating		None	None	3	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange.
Floorbeam	FB2	None	None	4	5	Localized section of web at connection to S0 has 24% section loss (ultrasonic test).
Coating		None	None	3	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange.
Floorbeam	FB3	None	None	3	4	Localized 2mm pitting at connecting angle to S2-1. Localized very severe corrosion and pitting of the web at connecting angle to S3-2 - 56% section loss (ultrasonic test).
Coating		None	None	3	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange.
Floorbeam	FB4	None	None	3	4	Very severe corrosion and knife-edging of bottom flange and gusset plate on west side at connection to bracing.
Coating		None	None	3	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange.
Floorbeam	FB5	None	None	4	5	Light to medium corrosion along bottom flange and base of web. Light corrosion on underside of top flange.
Coating		None	None	3	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange.
Floorbeam	FB6	None	None	4	5	Light to medium corrosion along bottom flange and base of web. Light

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						corrosion on underside of top flange.
Coating		None	None	2	3	Coating failure along base of web and bottom flange, and on underside of top flange. Coating on underside of bottom flange flaking/cracked along member.
Floorbeam	FB7	None	None	4	5	Light to medium corrosion along bottom flange and base of web. Medium corrosion on underside of top flange.
Coating		None	None	2	3	Coating failure along base of web and bottom flange, and on underside of top flange. Coating on underside of bottom flange flaking/cracked along member.
Floorbeam	FB8	None	None	3	4	Light to medium corrosion along edges of bottom flange and top and base of web. Medium corrosion on underside of top flange. Localized 100mm-long section of west web at connection to S7-2 has 27% section loss (ultrasonic test). Localized 100mm-long section of west bottom flange at connection to S7-2 has 22% section loss (ultrasonic test).
Coating		None	None	2	3	Coating failure along top and base of web, bottom flange, and on underside of top flange. Coating on underside of bottom flange is cracked at localized areas.
Floorbeam	FB9	None	None	4	5	Light to medium corrosion along edges of bottom flange and base of web. Medium corrosion on underside of top flange.
Coating		None	None	2	3	Coating failure along base of web and edges of bottom flange, and on underside of top flange. Coating on underside of bottom flange is cracked along member.
Stringers						
Stringer	S0-1	None	None	3	5	Medium corrosion on underside of bottom flange over 50% of west half. Severe corrosion on top flange of end stringer at connection to FB0. Light corrosion on 30% of interior web face. Medium corrosion with flaking steel in web near connection to FB0. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Supporting angle at FB0 is 6mm below bottom flange of stringer.
Coating		None	None	1	1	50% of coating has failed on west half of underside of bottom flange. 30% of coating on interior web face has flaked off.
Stringer	S0-2	None	None	3	5	Medium corrosion on 50% of underside of bottom flange. Severe corrosion on top flange of end stringer at connection to FB0. Rust jacking of angle supporting stringer at connection to FB1 has pushed angle down by 6mm. 3mm pitting and flaking steel at base of web at FB1. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer.
Coating		None	None	2	2	50% of coating has failed on underside of bottom flange.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Stringer	S0-3	None	None	3	4	Medium corrosion on 50% of underside of bottom flange. Severe corrosion on top flange and web of end stringer at connections to FB0 and FB1. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion over 70% of inside web.
Coating		None	None	1	1	50% of coating has failed on underside of bottom flange. 70% of coating on inside web failed.
Stringer	S0-4	None	None	3	4	Medium corrosion on 50% of underside of bottom flange. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer.
Coating		None	None	2	2	50% of coating has failed on underside of bottom flange.
Stringer	S0-5	None	None	3	4	Medium corrosion on 50% of underside of bottom flange. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer.
Coating		None	None	2	2	50% of coating has failed on underside of bottom flange.
Stringer	S0-6	None	None	3	4	Medium corrosion on 50% of underside of bottom flange. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer.
Coating		None	None	2	2	50% of coating has failed on underside of bottom flange.
Stringer	S1-1	None	None	3	4	Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S1-2	None	None	3	4	Medium corrosion and flaking rust on underside of top flange and at base of web at several locations. Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Several areas of coating failure on web and top flange.
Stringer	S1-3	None	None	3	4	Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S1-4	None	None	3	4	Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S1-5	None	None	3	4	Severe corrosion with flaking paint and deep pitting at base of web at east end of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S1-6	None	None	3	4	Severe corrosion with flaking paint and deep pitting at base of web at east end

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						of stringer. Medium corrosion along top and base of web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S2-1	None	None	3	4	Light corrosion on bottom 125mm of web at connection to FB3. 600mm-long section at base of north web has 23% section loss (ultrasonic test).
Coating		None	None	3	3	Coating flaked off from bottom 125mm of web at connection to FB3.
Stringer	S2-2	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S2-3	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S2-4	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S2-5	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S2-6	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S3-1	None	None	4	5	Light corrosion on bottom flange and bottom 125mm of web at connection to FB4. Light corrosion along length of bottom flange. Localized 2mm pitting at connecting angle to FB4.
Coating		None	None	3	3	Coating flaked off from bottom flange and bottom 125mm of web at connection to FB4. Coating failed along bottom flange.
Stringer	S3-2	None	None	3	5	Localized very severe corrosion and pitting of the web and bottom flange at connecting angle to FB3. Medium corrosion along top of web.
Coating		None	None	1	1	50% coating failure on web.
Stringer	S3-3	None	None	4	5	Extensive areas of light corrosion on web.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S3-4	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S3-5	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S3-6	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Stringer	S4-1	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S4-2	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S4-3	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S4-4	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S4-5	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S4-6	None	None	4	5	Extensive areas of light corrosion on underside of bottom flange.
Coating		None	None	2	2	Extensive areas of coating failure on underside of bottom flange.
Stringer	S5-1	None	None	4	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S5-2	None	None	4	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S5-3	None	None	3	4	Severe corrosion and rust jacking on bottom flange at east end. Supporting bracket on FB6 has severe corrosion and rust jacking. Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Coating failure on bottom flange at east end. Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S5-4	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S5-5	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S5-6	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-1	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-2	None	None	3	5	Gap between bottom flange and supporting bracket on FB6. Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-3	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-4	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-5	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S6-6	None	None	3	5	Gap between bottom flange and supporting bracket on FB6. Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-1	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.

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PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-2	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-3	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-4	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-5	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web. Localized 100mm-long section of south web at connection to FB8 has 33% section loss (ultrasonic test).
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S7-6	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-1	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-2	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-3	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.

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STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-4	None	None	3	5	Medium corrosion and pitting at base of web at FB9. Localized areas of coating failure on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	4	4	Coating loss at base of web at FB9. Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-5	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.
Stringer	S8-6	None	None	3	5	Localized medium corrosion on underside of bottom flange, and at top and base of web. Localized areas of 20% to 30% section losses at base of web.
Coating		None	None	3	3	Localized areas of coating failure on underside of bottom flange, and at top and base of web.

Secondary Components

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Bottom Chord Bracing						
Diagonal Bracing	0S-1N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	0N-1S	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	1S-2N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	1N-2S	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	2S-3N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	2N-3S	None	None	2	3	75mm x 50mm perforation and larger area of severe section loss in horizontal leg of member. Light corrosion over 10% of member.
Coating		None	None	2	2	Large areas of coating failure.
Diagonal Bracing	3S-4N	None	None	4	4	4mm pitting across underside of bracing member at connection to 3N-4S. Light corrosion over 10% of member.
Coating		None	None	2	2	Large areas of coating failure.
Diagonal Bracing	3N-4S	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	5S-6N	None	None	2	2	Severe corrosion and large (200mm long) perforation in horizontal leg at connection with 5N-6S.
Coating		None	None	1	3	Flaking coating over majority of top surface of horizontal leg and inside face of vertical leg.
Diagonal Bracing	5N-6S	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal Bracing	6S-7N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	1	3	Flaking/cracked coating over majority of bottom surface of horizontal leg.
Diagonal	6N-7S	None	None	2	4	Severe pitting and 3 small perforations in S/E horizontal leg. Medium

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STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Bracing						corrosion along length of horizontal leg.
Coating		None	None	1	1	100% coating failure on top surface of horizontal leg.
Diagonal Bracing	7S-8N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	Localized areas of coating failure (10%)
Diagonal Bracing	7N-8S	None	None	4	5	Severe pitting in underside of bottom flange at intersection with 7S-8N.
Coating		None	None	1	3	Coating on underside is cracked along length.
Diagonal Bracing	8S-9N	None	None	4	5	Light corrosion over majority of top surface of horizontal leg.
Coating		None	None	1	2	Majority of coating has failed on top surface of horizontal leg. Coating on underside is cracked along length.
Diagonal Bracing	8N-9S	None	None	4	5	Light corrosion over majority of top surface of horizontal leg.
Coating		None	None	1	2	Majority of coating has failed on top surface of horizontal leg. Coating on underside is cracked along length.
Upper Sway Bracing						
Portal Frame	1S-1N	None	None	4	4	Light corrosion over 30% of member. North diagonal members are bent. Water is ponding and moss is growing in bottom lateral member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	1S-2N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	1N-2S	None	None	4	4	Member is bent horizontally at 2S. Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	2S-3N	None	None	4	4	Member is bent vertically at 3N. Light corrosion over 50% of member.
Coating		None	None	1	1	50% of coating flaked off.
Diagonal Bracing	2N-3S	None	None	4	5	Light corrosion over 50% of member.
Coating		None	None	1	1	50% of coating flaked off.
Lateral Bracing	2S-2N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	3S-4N	None	None	4	5	Light corrosion over 70% of member.
Coating		None	None	1	1	70% of coating flaked off.

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STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Diagonal Bracing	3N-4S	None	None	4	4	Light corrosion over 70% of member. Member is bent vertically and horizontally at 3N.
Coating		None	None	1	1	70% of coating flaked off.
Lateral Bracing	3S-3N	None	None	4	5	Light corrosion over 30% of member. Moss growing on bottom lateral angles.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	4N-5C	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	4C-5N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	4C-5S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	4S-5C	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Lateral Bracing	4S-4N	None	None	4	4	Light corrosion over 20% of members. Moss growing on bottom lateral angle. Rust jacking on top horizontal plate plus light corrosion over 75% of plate.
Coating		None	None	1	1	20% of coating flaked off, 75% on top horizontal plate.
Diagonal Bracing	5S-6N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Diagonal Bracing	5N-6S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Lateral Bracing	5S-5N	None	None	4	4	Light corrosion over 20% of members, and over 100% of top plate. Top plate bent due to rust jacking.
Coating		None	None	1	1	20% of coating flaked off, 100% on top plate.
Diagonal Bracing	6S-7N	None	None	4	5	Member sagging. Light corrosion over 50% of member.
Coating		None	None	1	1	50% of coating flaked off.
Diagonal Bracing	6N-7S	None	None	4	5	Member sagging. Light corrosion over 50% of member.
Coating		None	None	1	1	50% of coating flaked off.
Lateral Bracing	6S-6N	None	None	4	5	Light corrosion over 15% of members. Moss growing on angles.

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PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	1	1	15% of coating flaked off. 90% cracked/flaking on underside of bottom lateral member.
Diagonal Bracing	7S-8N	None	None	4	4	Member slightly bent. Light corrosion over 50% of member.
Coating		None	None	1	1	50% of coating flaked off.
Diagonal Bracing	7N-8S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	30% of coating flaked off.
Lateral Bracing	7S-7N	None	None	4	5	Light corrosion over 30% of members. Moss growing on angles.
Coating		None	None	1	1	30% off coating cracked or flaking.
Portal Frame	8S-8N	None	None	4	4	Light corrosion over 50% of members. Moss growing on bottom plate and lateral angle. Water accumulating n bottom angle. Central vertical member is slightly bent.
Coating		None	None	1	1	50% of coating gone.

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PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Remaining Components

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Primary Components					
Waterway	None	None	5	3	Very severe erosion up to 175mm deep along central section of the swing pier walls. Up to 2.5m in length of undermining of timber cribs at the south-east corner of the swing pier.
Abutment Walls	None	None	5	6	West abutment wall is in good condition.
Deck	None	None	4	5	Timber beam at west end of deck is rotten and split along entire length and needs replacing. Central section of timber beam at east end of deck is rotten and split. South deck between FB4 and FB3 – top of 4 lateral members are splitting. Central deck between FB0 and FB1 – top 20 mm of 6 lateral members is rotten. North deck between FB8 and FB9 – 30mm deep check of 1 lateral member. Central section of deck is full of small holes, and light abrasion. Broken 2x4 midway between FB0 and FB1 and S0-3 and S0-4.
Piers	None	None	2	5	<u>Centre Swing Pier:</u> Top: Several very large areas of severe scaling, including along west half of rail and on east and west vertical faces of pier; area of medium scaling along west section of rail; area of cracked grout beneath east side of rail; several wide cracks. Sides: Several large areas of severe and very severe scaling, particularly at the top. Very severe erosion along the length of the pier at the waterline (approx 300mm high on the east side and 430mm high on the west side). The concrete below the waterline on the west side of the pier is easily chipped away. Other**:** The river bed on the west side of the pier is bedrock. <u>North Rest Pier</u>: Top: Numerous narrow – wide transverse cracks; spalls at edge in S/E corner; large area of severe scaling at S/E corner; several areas of ponding water along longitudinal centreline; N/E corner of 2nd section of slab from south has settled by 25mm; sagging of 32mm at centre of CJ between 3rd and 4th sections from south; wide map cracks over entire surface of wall on top of pier at north end. Concrete and block sides: Concrete blocks are typically 42” long by 36” high. Numerous large areas of severe and very severe scaling and spalled concrete in the sides of the concrete cap. Numerous wide vertical cracks in the side of the concrete cap generally at construction joints or at where two concrete blocks meet. Numerous long and narrow, but deep spalls/disintegration at the interface of the concrete cap and concrete blocks. The edges of many of the concrete blocks are rounded by erosion. Several large and deep spalls in the concrete blocks have exposed the steel

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					lifting hooks. The concrete blocks generally overhang the timbers below by 75mm. <u>Timber Cribbing:</u> West: On the west elevation of the pier only 4 timbers are visible at the centre of the pier. The top timber generally overhangs the timbers below by 50mm. The timbers on the west elevation are in generally good condition with no undermining noted. Some minor rotting was noted on the corners of several timbers. At 7m from the north end, a 50mm wide by 125mm deep gap was noted between the ends of adjacent timbers at the riverbed. East: The top timber generally overhangs the timbers below by 50mm. The timbers vary in size from 8” to 12”. The top timber at the north end of the pier is loose. The end 125mm of one of the timbers at the north end of the pier is rotten. The 2nd timber from bottom at the north end of the pier has several 50mm voids in. Several of the bottom timbers are rotten (one each at 5m, 7m and 12m from the north end) for a depth of up to 150mm. The 14th timber from the top at 18m from the north end is rotten. 225mm of the lowest crosstie timber at 25m from the north end has rotted away. Two crosstie timbers at 20m from the north end have rotted to a depth of 200mm, and one at 23m to a depth of 430mm. The end 300mm of the top timber at the south end is missing. The top of the top timber at 17m has split off. The ends of two crossties at 13m are rotten to a depth of 300mm, and the end of the top timber at 5m by 175mm. At 5m from the north end the 3rd timber from the top has spilt longitudinally, and the end 800mm of another timber has split off completely. <u>Other:</u> The steel ladder on the east side of the pier is severely damaged and should be replaced. The river bed to the east of the pier is covered in approximately 150mm of silt, with up to 600mm of silt on the west side. The river bed in the north-west corner has around 700mm of silt. Water depth on east side ranges from 6.73m (5m from north end) to 4.09m (25m from north end). Water depth on west side ranges from 1.30m (20m from north end) to 2.13m (5m from north end). The water depth at the north-east corner is 6.55m. <u>South Rest Pier</u>: Top: Several narrow – wide transverse cracks; large areas of medium to severe scaling; several areas of ponding water along longitudinal centreline; sagging of 25mm at centre of 2nd section from south; severe scaling and map cracks over entire surface of wall on top of pier at south end. Large cracks and a small tree growing between steel nosing plates at south end

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					Concrete and block sides: Concrete blocks are typically 42" long by 36" high. Numerous large areas of severe and very severe scaling and spalled concrete in the sides of the concrete cap. Numerous wide vertical cracks in the side of the concrete cap generally at construction joints or at where two concrete blocks meet. Numerous long and narrow, but deep spalls/disintegration at the interface of the concrete cap and concrete blocks. The edges of many of the concrete blocks are rounded by erosion. Several large and deep spalls in the concrete blocks have exposed the steel lifting hooks. There is a large void beneath the steel nosing plates at the south end. The concrete blocks generally overhang the timbers below by 75mm to 200mm. <u>Timber Cribbing:</u> West: On the west elevation of the pier only 3 timbers are visible at the centre of the pier. The top timber generally overhangs the timbers below by 50mm. The timbers on the west elevation are in generally good condition with no undermining noted. The ends of the crossties at the south end of the west side are generally rotten. East: The top timber generally overhangs the timbers below by 50mm. The timbers on the east elevation are in generally good condition. There is a 330mm gap between the cribbing and the riverbed at the south-east corner of the pier that tapers to 0mm over a length of approximately 2.5m. There are several 250mmx250mm voids (one each at 13m, 15m and 25m from the south end) in the cribbing where the ends of the longitudinal timbers have rotted away. <u>Other:</u> The river bed to the east of the pier is covered in large rocks and sections of concrete. Water depth on east side ranges from 4.17m (south end) to 2.08m (north end). Water depth on west side ranges from 2.44m (5m from south end) to 1.93m (15m from south end). <u>East Pier:</u> Base: Consists of grout/concrete filled bags with a concrete cap. No defects/undermining noted. Grout/concrete bags at base of pier are easily chipped away. River bed has up to 150mm silt over large rocks. Approx 150mm of concrete cap below water line at time of inspection. 2 un-armoured hydro cables at south end of pier on river bed. Concrete Cap: Numerous transverse cracks and areas of map cracking in inclined section, particularly at both north and south ends. Several areas of severe scaling

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					and disintegration on upper vertical shaft. Very long narrow areas of severe disintegration and spalling at interface of inclined section and lower pier shaft at north and south ends. Large delaminated area at base of inclined section at south-west corner. Horizontal cracks with efflorescence at N/W corner and south end of upper shaft. Efflorescence leaking from bottom edge of inclined section on west side.
Wearing Surface	None	None	4	3	North boards: 300mm of all boards at the west end are rotten. #5 boards (south board) at east and west ends are rotten, and need replacing. 75mm at the south edge of #5 sounds hollow and has light abrasion along entire length of the deck. Board #4 at east end is rotten and needs replacing. East end of board #4 between FB0 and FB1 is rotten. West end (100mm) of board #1 between FB1 and FB2 is rotten. 1500mm long section of boards #3, #4, #5 rotten at FB2. Plywood shim beneath wearing surface boards at east end of deck is rotten. South boards: 1st and 2nd # 2 boards from east are rotten and need replacing. 75mm at north edge of board #1 sounds hollow and has light abrasion along entire length of deck. 100mm long section of board #3 between FB6 and FB7 is rotten. 1000mm section of board #2 at FB5 is rotten. Board #5 between FB3 and FB4 is rotten and needs replacing. 1800mm long section of board #3 at FB3 rotten. 500mm and 700mm long sections of board #3 at FB1 rotten. 1800mm long section of board #2 between FB0 and FB1 rotten.
Structural Steel Coatings on Primary Components	None	None	1	1	The coating system is in poor condition throughout, with extensive areas of cracked and flaking coating typically noted, permitting corrosion to develop on the steel members. Laboratory tests on the coating system indicate that it contains levels of lead above current acceptable limits.
Pivot Structural Steel	None	None	3	4	Localized section of west bottom flange of girder beneath FB4 has 500mm-long section of 23% section loss (ultrasonic test). Coating has failed over 50% of girders, with light to severe corrosion, particularly on bottom flange. Severe section loss of bottom flange and rivet heads at connections with bracing members, including perforations on gusset plates. Ends of bracing severely corroded at connections. Central girder previously strengthened to repair cracks at bottom of web – cracks welded and vertical stiffeners added ~10 years ago. Localized section of north-west bottom flange of hub at pivot has 200mm-long section of 55% section loss (ultrasonic test). Localized section of bottom flange of member connecting 2 hub members at pivot has 150mmx50mm perforation. Top and bottom gusset plates connecting hub member and diagonal bracing at pivot has 50% section loss.
Secondary Components					
Embankments not Supporting	None	None	4	5	10% erosion at the end of and adjacent to the north-west wingwall due to water runoff from the roadway. There is also a large tree growing near the wingwall. No

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Remaining Components

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Foundations					erosion noted in the south-west embankments. Small trees growing in front of West Abutment wall. Some erosion of embankment material in front of West Abutment wall.
Ballast Walls	None	None	6	6	West ballast wall is in good condition. No significant defects other than a single vertical crack and a rust stain were noted.
Wingwalls	None	None	5	6	Small areas of light honeycombing and medium scaling on the north-west wingwall.
Bearing Seats	None	None	6	6	West bearing seat is in good condition. The east bearing seat on the east pier is in good condition. Accumulation of dirt and debris typically noted. No significant defects noted.
Joints	None	None	5	4	The joints at west and east end of bridge are open joints, allowing dirt, debris and rain/snow to fall onto the bearing seats.
Curbs	None	None	5	5	Light abrasion along the length of curbs, and minor splits and checks typical. Member at west end of north curb is loose and splitting longitudinally. The member between FB3 and FB4 on the south curb has a 6mm longitudinal split along its length. The north and south members at the east end are not tapered.
Approach Slabs	None	None	5	5	Areas of light ravelling at centreline and south side of approach wearing surface, and light abrasion on end dam. End dam is sloped to allow smooth passage onto bridge but creates uneven ride for vehicles.
Railings	None	None	3	3	The railings do not meet current CHBDC crash test requirements. North railing: Bottom connecting bolt at west side of FB8 is loose; bottom connecting bolt at east side of FB5 is loose; impact damage to 2 lattice at east of FB6, 1 lattice and baluster at FB5, the baluster at FB4, the bottom rail and baluster at FB3, the bottom rail between FB1 and FB2, and the end baluster between FB3 and FB4. South railing: Bottom connecting bolt at FB3 is loose; missing bolt on bottom rail west side of FB6; missing bolt at connection to post between FB8 and FB9; impact damage to bottom rail and lattice between FB1 and FB2, the bottom rail east of FB3, the baluster at FB5, the baluster at FB6, and 2 balusters at FB8. Coating has failed on at least 30% of lattice and at least 50% of top and bottom rails, with light to medium corrosion developing.
Approach railings	None	None	1	1	The railings do not meet current CHBDC crash test requirements. The steel cables on both the north and south sides of the west approach have tension loss. The west end of the steel cable on the south side of the approach is attached to a road sign post. The first 10 (ten) posts at the east end on the south side are completely rotten. The steel tube railing on the north side of the approach has slight impact damage and small areas of coating failure. The steel tube railing posts on the south side only have

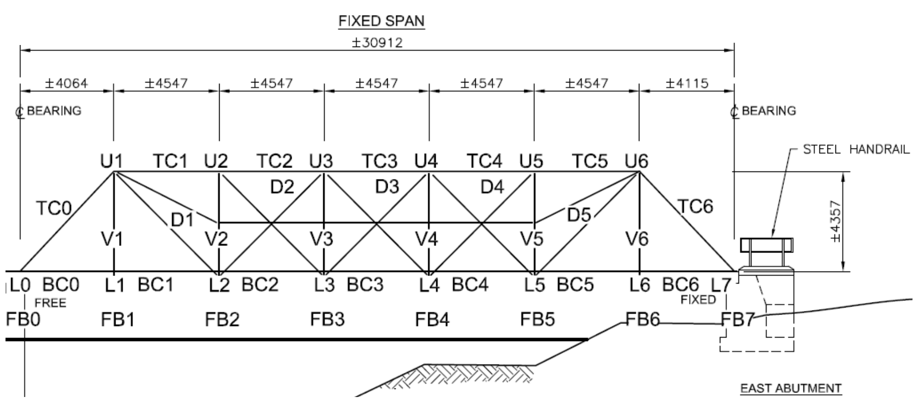
MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Swing Span
ELEMENTS: Remaining Components

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					2 of 4 anchor bolts installed.
Structural Steel Coatings on Secondary Components	None	None	1	1	The coating system is in very poor condition throughout, with extensive areas of cracked and flaking coating typically noted, permitting corrosion to develop on the steel members. Laboratory tests on the coating system indicate that it contains levels of lead above current acceptable limits.
Auxiliary Components					
Slope Protection	None	None	5	5	Some slope protection stones at the West Abutment embankment have been displaced.
Signs	None	None	N/A	N/A	The street name and traffic light sign posts on the west approach are not vertical. The bottom bolt is missing from the “slippery road” sign at the west end of the north truss. The “hazard close to edge of road” sign at the west end of the south truss is loose and has some impact damage. The “stop here on red signal” sign on west approach is loose.
Utilities	None	None	N/A	N/A	The light at the south-west corner of the truss is broken.

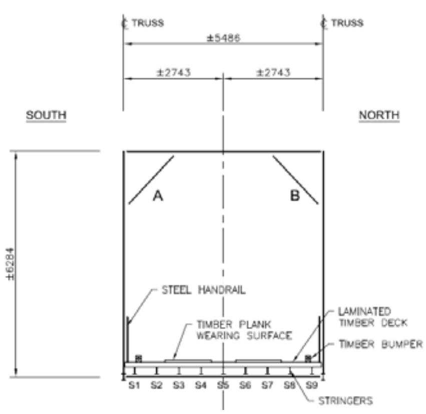
INSPECTION FORM

NAME:	Hamlet Bridge (Bridge 57) – Fixed Span
LOCATION:	Canning Road, Hamlet, Ontario
YEAR CONSTRUCTED:	Circa 1905-1922 *

SOUTH ELEVATION



SOUTH ELEVATION



CROSS-SECTION

Notes:

1. Steel through-truss simply-supported single span bridge.
2. Timber deck and timber plank wearing surface.
3. Concrete west pier with grout-filled bag sub-structure beneath the waterline.
4. Concrete east abutment.

* Superstructure built circa 1905, substructures built circa 1915-1922

INSPECTION FORM

NAME:	Hamlet Bridge (Bridge 57) – Fixed Span
LOCATION:	Canning Road, Hamlet, Ontario
YEAR CONSTRUCTED:	1905-1922
TYPE OF INSPECTION:	Comprehensive Detailed Inspection
Original Design:	Unknown
Drawings Available:	Yes
Previous Inspection Report Date:	None
Author:	N/A
Current Inspection Date:	September 28 and 29, 2011
Inspectors:	Patrick Mergel, P.Eng., ing.; Ben MacMaster, P.Eng.; Peter Harvey, EIT.
Temperature:	15°C-21°C (28th); 13°C-18°C (29th);
Weather:	Rain a.m., sunny p.m. (28th); Mainly cloudy, late thunderstorms (29th)
Equipment:	Dive boat supplied by Lower Lakes Marine; Pontoon boat supplied by Loon Wing Lift Services; Bucket truck supplied by Rostance Electric.
Previous Condition Rating:	None
Previous Functional Rating:	None
Current Structural Condition Rating:	2
Current Functional Rating:	2

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Waterway (P)	Minor erosion noted at east embankment.	5	D	
Foundations (P)	Gradual east abutment movements over time reported by bridge operator. Some out-of-plumbness noted. Possible distress on east abutment from bridge superstructure.	2	B	F5-F7
Abutments (P)	Condition rating based on PCR of abutment wall and wingwalls. Large areas of severe scaling, delaminations and spalling below the east abutment wall horizontal construction joint. The majority of the east bearing seat is covered in dirt and debris. The top of the abutment wall has tilted west towards the river, indicating that movement may have taken place. Several wide vertical cracks and areas of spalling and delaminated concrete noted in the east ballast wall. The north-east wingwall has wide gaps at the horizontal and vertical construction joints, with some vegetation growing through. Areas of spalling and disintegration, plus some wide cracks also noted. The south-east wingwall has wide gaps at the horizontal construction joints, an	2	B/M	F5-F9

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
	area of spalling/disintegration at the base of vertical construction joint, plus some medium cracks. The top of the south-east wingwall has tilted towards the south, indicating that movement may have taken place.			
Girders (Trusses) (P)	V1N is twisted about its longitudinal axis along its full length. V6N is bent at base due to impact damage. Localized area of 10% (south truss) to 30% section loss (north truss) in south flange at connection with diagonal member of the lateral bracing, and severe localized corrosion and section loss of inside flange at base of the members is typical. Coatings are typically cracking and peeling at base of member. At diagonal D2N, there is a turnbuckle splice near bottom of L2N-U3N, and impact damage to inside member of U2N-L3N. At diagonal D3N, there is a turnbuckle splice near bottom of member U3N-L4N. At diagonal D4N, there is a clamped splice at base of U4N-L5N. 5% of coating has typically flaked off. Extreme (>90%) section loss of the bottom chord I-bars at the east end of the north and south trusses with only approximately 1/16 of the original bottom bars remaining. Severe corrosion of the bottom bar of the bottom chord I-bars BC0 at west end of both north and south trusses, with localized 30% and 40% section loss respectively. Typical 30-40% corrosion of bottom I-bar member at most connections. 10% of coating has typically flaked off from the top plates channels on the top chords, allowing light corrosion to develop.	1	A (With urgent repairs already complete)	F12-F22
Floor System (P)	Condition rating based on floor beam MCR. Floor beams: Extensive areas of coating loss and light to very severe corrosion on majority of members. Severe localized section loss of many members, including top flanges at stringers, and webs near bottom flanges. Stringers: out-of-straightness noted in the stringers when viewed longitudinally from the east abutment. Approximately 20% of coatings have failed on each stringer, leading to light to medium corrosion. Some primer coat is also typically exposed.	3	B	F22,F23, F25

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Coatings (P, S)	The coatings are in generally poor condition, with extensive areas of cracking and flaking noted, permitting corrosion to develop on the steel members. Red lead primer observed and confirmed by testing. South truss coating is a darker shade of blue than the north truss coating.	1	B	F12,F13, F14,F19, F20, F21,F22, F23,F25
Deck (P)	Transverse beam at west end of deck is severely rotted. Light splitting and rotting at numerous locations, with some end splits noted. Accumulation of dirt and debris on north section.	5	B/M	F24,F26, F27,F28
Wearing Surface (P)	Minor splits, checks and wear typical along length of deck of north and south sections of wearing surface. The boards at the east and west ends of the deck are generally rotten. Numerous other boards along the length of the deck have long sections of severe rotting. Inside edges of the boards either side of the central longitudinal section of deck sound hollow and have light abrasion along entire length of the deck.	4	B	F24,F26
Pin and Hanger Bearings (P)	Very severe corrosion and section loss of the north and south pins and housing at L7. Ultrasonic testing indicated no cracks in bolts or pins at connections.	2	B	F12,F21, F22
Piers (P)	Observed that the grouted bags under the concrete pier are soft and easily chipped with a hammer. Numerous transverse cracks and areas of map cracking in the inclined section of the concrete pier , particularly at the north and south ends, and a large delaminated area at the base in the south-west corner. Very long narrow areas of severe disintegration and spalling at the interface of the inclined and vertical sections of the concrete pier. Efflorescence observed at the bottom edge of the inclined section on the west side. Several areas of severe scaling, severe disintegration and horizontal cracks with efflorescence on the upper vertical shaft.	4	B	F10, F11
Curbs (S)	Minor abrasion, checks and splits typical on both curb faces. Several sections have severe longitudinal splits. East end of north and south curbs are rotten. Spacer block beneath south curb at west end has split into two pieces.	4	B	F27,F28
Bottom Chord Bracing (S)	Extensive areas of coating failure, with large areas of exposed primer and light	4	D	F23

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
	to medium corrosion typical.			
Upper Sway Bracing (S)	Impact damage has shifted bottom lateral member of west portal frame up and to the east by around 150 mm, and top lateral member is bent at south end. Bottom flange of top lateral member of east portal frame is bent at south end. Up to 30% of coatings have flaked off leading to light corrosion.	3	B	F13
Deck Joints (S)	The open joints at the west and east ends of the bridge are allowing dirt, debris and rain/snow to fall onto the bearing seats.	3	D	F8, F24
Approaches (S)	Wide transverse cracks at the east end of the east approach, areas of ponding, and several asphalt patch repairs.	4	B	F4
Railings (S)	The bridge railings do not meet current CHBDC crash-tested standards for bridge barriers. Coatings have failed on at least 50% of the original post and lattice railing system with extensive light corrosion. The south panel is bent around the end diagonal. Several angles and hooks connecting the steel tube and cable railing system to the trusses are bent. Small localized areas of coating failure with light corrosion. Several locations of minor impact damage to the top rail.	4	B	F10, F33
Guiderails (S)	The guiderails do not meet current MTO standards. The timber posts and steel cable railings on the east approach are in poor condition. The north-east and south-east cables are not tight. The two end posts are rotten on both the north-east and south-east railings.	1	A	F34
Bearings (S)	The north-west and south-west roller bearing assemblies are both out of alignment and off the bearing plates.	2	B	F29, F30
Embankments (S)	Severe erosion of the north-east embankment at the end of the wingwall due to water run-off has eroded the soil around the end post of the steel cable railing.	2	A	F4,F6,F7, F32,F34
Slope Protection (A)	The majority of slope protection stones in front of the east abutment sheet piling have been washed away.	1	A	F7

ELEMENT	OBSERVATION	CONDITION RATING	PRIORITY CODE	PHOTO NO.
Signs (A)	The "Slippery road" at the south-east corner of the truss has impact damage and is also loose. Impact damage to the "hazard close to edge of road" sign at the south-east corner of truss.	N/A	A/M	F4
Utilities (A)	There are two un-armoured cables on the river bed at the south end of the pier.	N/A	D	

RECOMMENDATIONS

1. Replace or repair the north and south bottom chord members at the east end of the bridge (Temporary repairs already completed, permanent repairs to be completed within 1 year). Consider replacing the entire bottom chord if the current bridge is to be left in service for an extended period of time. Replace the entire bottom chord within 3 years in any case.
2. Replace the steel cable guiderail in the east approach (1 year).
3. Replace the eroded material at the north-east embankment. Add slope protection measures such as stone rip-rap (1 year).
4. Replace the displaced slope protection stones at the east abutment (1 year).
5. Replace the damaged signs and secure loose signs (1 year).
6. Patch the depressions in the east approach wearing surface asphalt wearing surface to prevent ponding, and rout and seal cracks (1 year).
7. Blast-clean and re-coat the structural steel (3 years).
8. Replace the roller bearings at the west end of the bridge (3 years).
9. Replace deteriorated areas of the timber deck and curbs (3 years).
10. Replace the timber wearing surface (3 years).
11. Repair or replace the impact-damaged portal frame members (3 years).
12. Perform concrete repairs and crack injections on the pier (3 years). Consider re-facing the pier for a more complete rehabilitation that will reduce future repair contracts and avoid a "patchwork" appearance.
13. Replace the north and south pins and housing at L7 (3 years).
14. Perform concrete repairs and crack injection and/or re-facing at the east abutment (3 years). If the bridge is to be replaced, recommended to completely re-face the east abutment or replace the abutment entirely.
15. Flush dirt and debris from bearing seats and deck (on-going).

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Top Chord						
Truss	TC0	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC1	None	None	4	5	Light corrosion over 10% of member. Bird nest on bottom plate at west end.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC2	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of top plate coating has flaked off.
Truss	TC3	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of top plate coating has flaked off.
Truss	TC4	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of top plate coating has flaked off.
Truss	TC5	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of top plate coating has flaked off.
Truss	TC6	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Bottom Chord						
Truss	BC0	None	None	3	4	Very severe corrosion and 30% section loss of the bottom section of the bottom chord I-bar members at west end.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC1	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC2	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC3	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC4	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC5	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC6	None	None	1	1	Extreme (>90%) section loss of bottom chord members at east end – only 1/16 of original section remains.
Coating		None	None	4	4	10% of coating has flaked off.
Intermediate Chord						
Truss	IC1	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.
Truss	IC2	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC3	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC4	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC5	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.
Diagonals						
Truss	D1	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Truss	D2	None	None	4	5	Turnbuckle splice near bottom of member L2-U3. Light corrosion over 5% of member. Impact damage to inside member of U2-L3.
Coating		None	None	5	5	5% of coating has flaked off.
Truss	D3	None	None	5	5	Turnbuckle splice near bottom of member U3-L4. Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.
Truss	D4	None	None	5	5	Clamped splice at base of U4-L5. Light corrosion at top connection.
Coating		None	None	5	5	Coating is flaking at top connection.
Truss	D5	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Verticals						
Truss	V1	None	None	3	4	Member is twisted about longitudinal axis along full length. Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V2	None	None	4	5	Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V3	None	None	4	5	Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V4	None	None	3	4	Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Localized area of 70% section loss of south-east flange at base of member (ultrasonic test).
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V5	None	None	4	5	Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Localized area of 35% section loss of south-east flange at base of member (ultrasonic test).
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V6	None	None	3	4	Localized area of 30% section loss in south flange at connection with diagonal member of the lateral bracing. Member is bent at base due to previous impact damage. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Lower Connections						
Truss	L0	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L1	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L2	None	None	3	5	Light to medium corrosion over 50% of connection.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: North Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L3	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L4	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L5	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L6	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L7	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Upper Connections						
Truss	U1	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U2	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U3	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U4	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U5	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U6	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Top Chord						
Truss	TC0	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC1	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC2	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC3	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC4	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC5	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	TC6	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Bottom Chord						
Truss	BC0	None	None	3	4	Very severe corrosion and 40% section loss of the bottom section of the bottom chord I-bar members at west end.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC1	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC2	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC3	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC4	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Truss	BC5	None	None	4	5	Medium to severe corrosion at connections. Light corrosion over 10% of member.
Coating		None	None	4	4	10% of coating has flaked off.
Truss	BC6	None	None	1	1	Extreme (>90%) section loss of bottom chord members at east end – only 1/16 of original section remains.
Coating		None	None	4	4	10% of coating has flaked off.
Intermediate Chord						
Truss	IC1	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.
Truss	IC2	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC3	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC4	None	None	5	5	Light corrosion over 2% of member.
Coating		None	None	5	5	2% of coating has flaked off.
Truss	IC5	None	None	5	5	Light corrosion over 5% of member.
Coating		None	None	5	5	5% of coating has flaked off.
Diagonals						
Truss	D1	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Truss	D2	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Truss	D3	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Truss	D4	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Truss	D5	None	None	5	5	Light corrosion over less than 5% of member.
Coating		None	None	5	5	Less than 5% of coating has flaked off.
Verticals						
Truss	V1	None	None	4	5	Localized area of 10% section loss in south flange at connection with

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V2	None	None	4	5	Localized area of 10% section loss in south flange at connection with diagonal member of the lateral bracing. Localized area of severe section loss of the north-west flange at the base of the member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V3	None	None	4	5	Localized area of 10% section loss in south flange at connection with diagonal member of the lateral bracing. Localized area of 9% section loss of north-east flange at base of member (ultrasonic test).
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V4	None	None	4	5	Localized area of 10% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V5	None	None	4	5	Localized area of 10% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Truss	V6	None	None	4	5	Localized area of 10% section loss in south flange at connection with diagonal member of the lateral bracing. Severe localized corrosion and section loss of inside flange at base of member.
Coating		None	None	4	4	Coating is cracking and peeling at base of member.
Lower Connections						
Truss	L0	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L1	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L2	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L3	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: South Truss

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Truss	L4	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L5	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L6	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Truss	L7	None	None	3	5	Light to medium corrosion over 50% of connection.
Coating		None	None	1	1	50% of coating has flaked off.
Upper Connections						
Truss	U1	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U2	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U3	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U4	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U5	None	None	5	5	Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate, 5% overall.
Truss	U6	None	None	4	5	Top plate is severely bent due to rust jacking. Light corrosion over 20% of top plate, 5% overall.
Coating		None	None	5	5	Coating has flaked off over 20% of top plate.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Floor Systems and Bracing

Primary Components

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Floorbeams						
Floorbeam	FB0	None	None	3	5	Medium to severe corrosion along flanges and web.
Coating		None	None	1	1	Extensive areas of coating failure.
Floorbeam	FB1	None	None	3	5	Medium to severe corrosion along flanges and web.
Coating		None	None	1	1	Extensive areas of coating failure.
Floorbeam	FB2	None	None	3	4	Medium to severe corrosion along flanges and web. Localized section of east top flange at connection to S7 has 50% section loss (ultrasonic test).
Coating		None	None	1	1	Coating has failed over 40% of member.
Floorbeam	FB3	None	None	3	4	Medium to severe corrosion along flanges and web. Localized section of west web at connection to S3 and S4 has 58% section loss (ultrasonic test).
Coating		None	None	1	1	Coating has failed over majority of member.
Floorbeam	FB4	None	None	3	4	Medium to severe corrosion along flanges and web. Several localized sections of east web at connection to stringers have up to 45% section loss (ultrasonic test).
Coating		None	None	1	1	Coating has failed over majority of member.
Floorbeam	FB5	None	None	3	4	Medium to severe corrosion along flanges. Localized section of west bottom flange has 33% section loss (ultrasonic test).
Coating		None	None	1	1	Coating failed on flanges.
Floorbeam	FB6	None	None	3	4	Medium to severe corrosion along flanges. 80mm long section of west bottom flange has 35% section loss (ultrasonic test).
Coating		None	None	1	1	Coating failed on over 50% of member.
Floorbeam	FB7	None	None	3	5	Medium to severe corrosion along flanges and web.
Coating		None	None	1	1	Extensive areas of coating failure.
Stringers						
Stringer	S1	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S2	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S3	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20%

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						of coating on flanges has failed.
Stringer	S4	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S5	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S6	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S7	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S8	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.
Stringer	S9	None	None	4	5	Stringer is bent along length. Light to medium corrosion on 20% of flanges.
Coating		None	None	3	3	Areas of coating failure along base of webs has exposed primer coat. 20% of coating on flanges has failed.

Secondary Components

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Bottom Chord Bracing						
Diagonal Bracing	0S-1N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	0N-1S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Diagonal Bracing	1S-2N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	1N-2S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	2S-3N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	2N-3S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	3S-4N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	3N-4S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	4S-5N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	4N-5S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating has failed, with large areas of exposed primer and corrosion.
Diagonal Bracing	5S-6N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating is flaking or cracked, with large areas of exposed primer and corrosion.
Diagonal Bracing	5N-6S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating is flaking or cracked, with large areas of exposed

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
						primer and corrosion.
Diagonal Bracing	6S-7N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating is flaking or cracked, with large areas of exposed primer and corrosion.
Diagonal Bracing	7N-7S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	At least 50% of coating is flaking or cracked, with large areas of exposed primer and corrosion.
Upper Sway Bracing						
Portal Truss	1S-1N	None	None	3	3	Impact damage has shifted bottom lateral member up and to the east by around 150mm. Light corrosion over 20% of members. Top lateral member is bent at south end.
Coating		None	None	1	1	Coating has flaked off over 20% of members.
Diagonal Bracing	1S-2N	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Diagonal Bracing	1N-2S	None	None	4	5	Light corrosion over 20% of member. Localized areas of medium corrosion on underside of member at end.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Lateral Bracing	2S-2N	None	None	4	5	Light corrosion over 10% of member.
Coating		None	None	4	4	Coating has flaked off over 10% of member.
Diagonal Bracing	2S-3N	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Diagonal Bracing	2N-3S	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Lateral Bracing	3S-3N	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Diagonal Bracing	3S-4N	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Diagonal Bracing	3N-4S	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Floor Systems and Bracing

Element	Member	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Lateral Bracing	4S-4N	None	None	4	5	Light corrosion over 10% of member. West flange is deformed along length.
Coating		None	None	4	4	Coating has flaked off over 10% of member.
Diagonal Bracing	4S-5N	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Diagonal Bracing	4N-5S	None	None	4	5	Light corrosion over 20% of member.
Coating		None	None	1	1	Coating has flaked off over 20% of member.
Lateral Bracing	5S-5N	None	None	4	5	Rust jacking at connection of diagonal member and V5 has bent connecting plate. 10% localized section loss in top lateral member at interface with south top chord.
Coating		None	None	1	1	Coating has flaked off over 30% of member.
Diagonal Bracing	5S-6N	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	Coating has flaked off over 30% of member.
Diagonal Bracing	5N-6S	None	None	4	5	Light corrosion over 30% of member.
Coating		None	None	1	1	Coating has flaked off over 30% of member.
Portal Truss	6S-6N	None	None	3	5	Bottom flange of top lateral member is bent at south end. Light corrosion over 10% of member.
Coating		None	None	4	4	Coating has flaked off over 10% of member.

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Remaining Components

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
Primary Components					
Waterway	None	None	5	5	Minor erosion noted at east embankment.
Foundations	None	None	N/A	2	Suspected movement of the east abutment due to overstressing of founding soils beneath shallow foundation.
Abutment Walls	None	None	4	2	Large areas of severe scaling and spalling below horizontal construction joint. East bearing seat completely covered in dirt and debris. The top of the abutment wall has tilted west towards the river, indicating that movement may have taken place.
Pin and Hanger Bearings	None	None	2	5	Very severe corrosion and section loss of north and south pins and housing at L7. No evidence of cracked bolts and pins at connections were found by the ultrasonic testing.
Deck	None	None	5	5	Entire transverse beam at west end of deck is rotten. Small holes along central section. Splits and light rotting at numerous locations, accumulations of dirt and debris. Some end splits in members.
Pier	None	None	4	5	Base: Consists of grout/concrete filled bags with a concrete cap. No defects/undermining noted. Grout/concrete bags at base of pier are easily chipped away. River bed has up to 150mm over large rocks. Approx 150mm of concrete cap below water line at time of inspection. Concrete Cap: Numerous transverse cracks and areas of map cracking in inclined section, particularly at both north and south ends. Several areas of severe scaling and disintegration on upper vertical shaft. Very long narrow areas of severe disintegration and spalling at interface of inclined section and lower pier shaft at north and south ends. Large delaminated area at base of inclined section at south-west corner. Horizontal cracks with efflorescence at N/W corner and south end of upper shaft. Efflorescence leaking from bottom edge of inclined section on west side.
Wearing Surface	None	None	4	3	North: West 500mm of planks #2 and #3 (from north) are rotten. West 700mm of plank #4 (from north) are rotten Edge 3" of south plank rotten and split along length of deck. Board #5 at FB1 has rotten section 600mm long. Board #5 at FB6 has rotten section 600mm, board #3 at FB6 has rotten section 200mm long. Minor splits, checks and wear typical along length of deck. South: Edge 3" of north plank rotten and split along length of deck. East 400mm of boards #2 and #3 are rotten. Board #4 between FB5 and FB6 has large rotten area 1250mm long with checking. Boards #2 and #3 between FB2 and FB3 have rotten areas of 500mm long and 1500mm long respectively. 500mm long rotten section at

MCR / PCR FORMS
PROJECT TITLE & NUMBER: Comprehensive Detailed Inspections of Bridges in Central Ontario - PCA Project No. 2011-4650-20027340
STRUCTURE: Hamlet Bridge (Bridge 57) – Fixed Span
ELEMENTS: Remaining Components

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					FB4 of plank #5, and 800mm rotten section of plank #2 between FB1 and FB2. Minor splits, checks and wear typical along length of deck.
Structural Steel Coatings on Primary Components	None	None	1	1	The coating system is in localized poor condition throughout, with extensive areas of cracked and flaking coating typically noted, permitting corrosion to develop on the steel members. Laboratory tests on the coating system indicate that it contains levels of lead above current acceptable limits.
Secondary Components					
Embankments not Supporting Foundations	None	None	2	5	Severe erosion of north-east embankment at end of wingwall due to water run-off has eroded soil around end post of steel cable railing
Ballast Walls	None	None	4	5	Several wide vertical cracks and areas of spalling and delaminated concrete.
Wingwalls	None	None	4	2	North-east: Wide gaps at horizontal and vertical construction joints, with some vegetation growing through. Areas of spalling and disintegration, plus some wide cracks. South-east: Wide gaps at horizontal construction joints. Area of spalling/disintegration at base of vertical construction joint, plus some medium cracks. The top of the south-east wingwall has tilted towards the south, indicating that movement may have taken place.
Bearings	None	None	2	2	North-west and south west roller bearing assemblies are broken and the west roller is not on either top or bottom plates.
Joints	None	None	3	4	The joints at the west and east ends of the bridge are open joints, allowing dirt, debris and rain/snow to fall onto the bearing seats.
Curbs	None	None	4	4	Minor abrasion, checks and splits typical on both curb faces. North: West of FB2, 25mm wide end split – replace member. End 400mm at east end is rotting. South: east end has almost entirely rotted away around anchor bolt and should be replaced. 25mm wide split in curb member to west of FB3, impact damage to bolt - replace member. Spacer block beneath curb at west end has split into two pieces.
Approach Slabs	None	None	4	5	Wide transverse cracks at east end of east approach, areas of ponding, and several asphalt patch repairs.
Railings	None	None	4	4	The railings do not meet current CHBDC crash-test requirements. With lattice: Light corrosion and coating failure over 50% of area. South panel is bent

Element	Prev. MCR	Prev. PCR	New MCR	New PCR	Comments
					around end diagonal. Hook connecting bottom rail to D6 south has severe impact damage. Angles connecting top and bottom north rails to V6 are bent. Steel tube and steel cable: angles connecting south cable to post at FB2 and FB5 are bent; angles connecting north rails to post at FB3 and FB6 are bent; 2% coating failure with light corrosion; minor impact damage to top rail at some locations.
Approach railings	None	None	1	1	The railings do not meet current CHBDC crash-test requirements. The timber posts and steel cable railings on the east approach are in poor condition. The north-east and south-east cables are not tight. The two end posts are rotten on both the north-east and south-east railings.
Structural Steel Coatings on Secondary Components	None	None	1	1	The coating system is in localized poor condition throughout, with extensive areas of cracked and flaking coating typically noted, permitting corrosion to develop on the steel members. Laboratory tests on the coating system indicate that it contains levels of lead above current acceptable limits.
Auxiliary Components					
Slope Protection	None	None	1	5	The majority of slope protection in front of the east abutment sheet piling has been washed away.
Signs	None	None	N/A	N/A	Impact damage to “Slippery road” on south-east corner of truss, sign is also loose. Impact damage to “hazard close to edge of road” sign at south-east corner of truss.
Utilities	None	None	N/A	N/A	2 un-armoured cables at south end of pier on river bed.

APPENDIX B

STRUCTURAL INSPECTION PHOTOGRAPHS

Photo S1: North elevation.

Photo S2: South elevation.

Photo S3: South elevation.

Photo S4: Looking east from the west approach.

Photo S5: West approach. Note leaning traffic light and road signs.

Photo S6: East abutment/pier.

Photo S7: East pier. Note the typical cracks with efflorescence and areas of spalling and disintegration.

Photo S8: West elevation of the north section of the rest pier.

Photo S9: West elevation of the south section of the rest pier.

Photo S10: Top of the south section of the rest pier. Note the areas of ponding water.

Photo S11: Top of the north section of the rest pier.

Photo S12: Typical wide transverse cracks in the top of the rest pier concrete cap.

Photo S13: East side of the south section of the rest pier. Typical wide cracks and large spalled areas in the concrete cap and blocks.

Photo S14: South-east corner of the rest pier. Typical wide cracks and large spalled areas in the concrete cap and blocks. Note the void beneath the steel plates.

Photo S15: The ladder on the east side of the north section of the rest pier is bent in the downstream direction.

Photo S16: Large spalled section in the concrete cap on the west side of the south section of the rest pier.

Photo S17: South-east corner of the central swing pier. Very severe erosion of the walls at the waterline.

Photo S18: North-west corner of the central swing pier. Very severe erosion of the walls at the waterline, and several large areas of disintegration.

Photo S19: The west abutment is in good condition. Note the minor erosion of the embankment.

Photo S20: The north-west wingwall exhibits areas of honeycombing and scaling.

Photo S21: Looking east. The truss and bracing members typically have extensive areas of coating failure and light corrosion.

Photo S22: V6N – typical condition of vertical truss members with extensive areas of coating failure and light corrosion.

Photo S23: The east portal frame members exhibit the typical coating loss and light corrosion. Note the water collecting in the bottom member.

Photo S24: The upper lateral bracing members are typically bent.

Photo S25: The north diagonal member in the west portal frame is bent.

Photo S26: Looking east from the west embankment. Note the typical condition of the floor system and deck members.

Photo S27: South side of stringer S0-3 between F0 and FB1. Typical condition of stringers with coating loss and light to medium corrosion.

Photo S28: Typical light to medium corrosion of floor beams and stringers at floor system connections.

Photo S29: Typical gap between the supporting angle and the stringer bottom flange.

Photo S30: The east side of the web of FB3 has severe localized section loss at the connection with stringer S3-2.

Photo S31: The south-east section of bracing member 6N-7S has severe pitting and three small perforations.

Photo S32: Bracing member 5S-6N has a long perforation in the horizontal leg.

Photo S33: Very severe section loss and 75 mm x 50 mm perforation in bracing member 2N-3S.

Photo S34: Typical condition of the coating system and truss members at the lower connections.

Photo S35: The top layers of steel of the bottom section of vertical bracing between V4S and V5S have delaminated.

Photo S36: West side of V5S with extensive coating loss.

Photo S37: The steel members in the central pivot area typically have severe section loss, including perforations in gusset plates.

Photo S38: The steel members in the central pivot area typically have severe section loss, including perforations in gusset plates.

Photo S39: Typical severe section loss of pivot girder bottom flange.

Photo S40: Central section of exposed deck between FB0 and FB1. Several deck members have areas of rotting. Note the typical small rot holes throughout and the abrasion on the edges of the inside wearing surface members.

Photo S41: East end of central exposed section of deck. The end deck member is rotten and splitting. The plywood shims beneath the wearing surface members are also rotten.

Photo S42: The edge of the inside wearing surface boards are typically rotten.

Photo S43: West end of north curb. The curb is loose and is starting to split from the west end.

Photo S44: North railing at FB5. Impact damage to end baluster and lattice. The bolt in the lower connection is loose. Note typical coating system condition.

Photo S45: Missing bolt in bottom south rail between FB8 and FB9.

Photo S46: Impact damage to bottom rail on north railing between FB1 and FB2.

Photo S47: The north railing post between FB3 and FB4 is bent.

Photo S48: The steel cable in the north-west approach guide rail has tension loss.

Photo S49: The first ten posts at the east end of the south-west guiderail are rotten (east post shown).

Photo S50: The west end of the steel cable of the south-west guiderail is attached to a road sign post.

Photo S51: The sign on the west end of the north truss is missing a bolt.

Photo F1: South elevation.

Photo F2: Partial north elevation showing both bridge spans.

Photo F3: Looking east. Note the typical condition of the coating on the end top chord members.

Photo F4: East approach.

Photo F5: The north-east wingwall has wide gaps at the construction joints plus areas of spalling and disintegration.

Photo F6: The south-east wingwall has wide gaps at the construction joints plus areas of spalling and disintegration. The wall is leaning to the south.

Photo F7: East abutment exhibits large areas of spalling and severe scaling. The wall is leaning to the west.

Photo F8: Asphalt patch repairs in the east approach.

Photo F9: The east ballast wall has several vertical cracks.

Photo F10: East face of the pier – numerous cracks, spalls and areas of disintegration noted.

Photo F11: Typical very wide cracks, efflorescence and spalls in the south end of the pier.


Photo F12: Severe rust jacking of the top chord cover plate at U6S.

Photo F13: West horizontal flange of the lateral bracing member 4S-4N is deformed along its length.

Photo F14: V5N – typical localized section loss of the interior flange of the vertical members at the connection to the vertical bracing.

Photo F15: V6N is bent at the base.

Photo F16: V1N is twisted about its longitudinal axis.

Photo F17: Inside member of U2N-L3N is bent.

Photo F18: Splice in member U4N-L5N.

Photo F19: North bottom chord I-bar at east end of bridge has lost more than 90% of its lower cross-section.

Photo F20: Very severe section loss in the south bottom chord I-bars at the east end of bridge.

Photo F21: Severe corrosion and rust jacking of the north bottom chord I-bars at the west end of bridge.

Photo F22: Typical condition of the lower truss connections and floor beams.

Photo F23: Typical condition of the floor beams, stringers and bottom lateral bracing.

Photo F24: West end of the fixed span. Note the deteriorated timber deck member.

Photo F25: Looking east along the stringers. Some stringer out-of-straightness observed.

Photo F26: Typical area of rotting and checking in the timber wearing surface.

Photo F27: Severe rotting of the east end of the south curb.

Photo F28: The block beneath the west end of the south curb has split into two pieces.

Photo F29: South-west bearing – the roller has twisted diagonally and is partially off the steel bearing plate.

Photo F30: North-west bearing – the roller is no longer on the steel bearing plate.

Photo F31: Stringer bearings at east end of bridge.

Photo F32: Erosion of the north-east embankment material around the guiderail post.

Photo F33: Typical extensive coating failure and light corrosion on the decorative railing panels

Photo F34: The two end posts of the north-east guide rail are rotten.

APPENDIX C

NON-DESTRUCTIVE TESTING REPORTS

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October 17th, 2011

Delcan Corporation
1223 Michael Street
Suite 100
Ottawa, Ontario
K1J 7T2

Attention: Mr. Peter Harvey B.A. Sc, E.I.T.

Our File: 11-09-69-UT

Subject: Hamlet, Ontario Swing Bridge #57
Parks Canada Agency Project #: 2011-4650-20027340

1.0 Scope:

1.1 This report covers the structural steel corrosion survey inspection of the above noted swing bridge structure, located in Hamlet, Ontario.
The inspection consisted of visual inspection, ultrasonic thickness testing of representative members where deemed necessary / as directed by client. Testing of members was limited due to time constraints and unprepared surfaces.
Inspection date: September 28th & 29th, 2011

2.0 Observations:

Please note: The items listed herein are references to the attached inspection report pages and digital photographs as applicable.

2.1 Stringer(s) exhibited severe corrosion as noted on ultrasonic report and photo #'s 1, 2, 3 & 14.

2.2 The cantilever bridge "hub" structure exhibited severe corrosion / total loss as noted on the ultrasonic inspection report and photo #'s 5 & 6. Also per visual inspection report and photo #'s 19, 20 & 21.

2.3 Floor braces exhibited moderate to severe corrosion as per ultrasonic report and photo #'s 4, 7, 8, 9, 10, 11, 12, 13 & 15.

2.4 Diagonal bracing member detected in 'delaminated condition' as per visual report and photo # 16.

2.5 Cross / wind bracing detected with severe corrosion / total loss of member, as per visual report and photo #'s 17 & 18.

2.6 Thickness readings obtained were isolated to areas exhibiting greater than 10% (visual) loss. Access and time constraints limited the number of readings obtained. All accessible corrosion areas were recorded for client review / evaluation.

3.0 Results:

3.1 The attached ultrasonic thickness readings, visual inspection reports and magnetic particle inspection reports pages 1 through 10 inclusive conclude this report.

Should you have any further questions regarding this report, please do not hesitate to contact this office.

Inspected by: David Guest C.E.T.

Submitted by: ____________________

Alastair Aitken
CSA W178.2 Level III

CANADIAN WELDING BUREAU
519
A. AITKEN
LEVEL III
CSA W178.2
CERTIFIED WELDING INSPECTOR

Q.A. Manager
C.B. Non-Destructive Testing Ltd.

Attachments: Inspection reports (10 pages)

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET SWING BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Top Chord TC7 (North) at U8	Top flange	10.0-10.2		Reference UT	
	Web	6.6		Reference UT	
Diagonal D7 (North) at U8	Web	7.3-7.6		Reference UT	
Top Chord TC7 (South) at U8	Strut / flange lateral connector plate	9.3-9.7		Reference UT	
Top Chord TC7 (South)	Cross brace angle at U8	8.2-8.6		Reference UT	
Sway Brace VB6	Angle 'H'	8.4-9.0		Reference UT	
Wind Brace at TC5 elevation	VB5 (South to VB6 North)	8.0-8.2		Surface corrosion <10%	
Top Chord TC5 (North) at U6	Web (South)	10.5-10.7		Reference UT	
Diagonal D5 (North) at U6	Web (South)	13.3-13.6		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
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2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

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Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Post V6 (North)	Flange (South East)	10.3		Reference UT	
	(South West)	11.3		Reference UT	
Strut at Top Chord U4	Angle (West)	8.1-8.5		Reference UT	
Wind Brace at Top Chord TC3	Angle U3 (South) at U4 (North)	8.7-9.0		Reference UT	
Top Chord (South) at U4	Connector pl.	8.8		Reference UT	
	Web	11.3		Reference UT	
	Top Flange	10.7		Reference UT	
Bracing at V2 Post (South)	Section 'L' (Angle)	9.2-9.5		Reference UT	
Diagonal D1 (South) 'middle'	Web (North)	6.5		Reference UT	
Wind Brace at Top Chord	Angle U2 (North) to U1 (South)	8.2		Reference UT	

REMARKS:

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JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
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Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: 45° ☐ 60° ☐ 70° ☐ **OTHER** ☐

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Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Diagonal D3 (South) 'middle'	Web (North)	13.5		Reference UT	
Top Chord at U5 (middle)	Strut conn. pl.	7.5-8.1		Reference UT	
Top Chord at U7 (South)	Truss/strut conn. pl.	8.1-8.4		Reference UT	
Diagonal D8 (North)	Top flange	9.6		Reference UT	
	Web (South)	10.3		Reference UT	
Post V8 (North)	Web	10.5		Reference UT	
Diagonal D7 (South)	Web (North)	7.6		Reference UT	
Post V7 (South)	Web	9.9		Reference UT	
Diagonal D6 (North)	Web (South)	6.5		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: 45° ☐ 60° ☐ 70° ☐ OTHER ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Diagonal D6	Web (North)	6.2		Reference UT	
Post V6 (North)	Web	10.0		Reference UT	
Diagonal D5 (South)	Web (North)	12.3		Reference UT	
Diagonal D5 (North)	Web (South)	13.2		Reference UT	
Post V5 (North)	Web	10.5		Reference UT	
Diagonal D3 (North)	Web (South)	12.4		Reference UT	
Diagonal D3 (South)	Web (North)	12.6		Reference UT	
Post V3 (South)	Web	10.8		Reference UT	
Post V3 (South)	Web	10.9		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1			Remarks:	Photo #:
Diagonal D2 (South)	Web (North)	6.2				Reference UT	
Diagonal D2 (North)	Web	6.3				Reference UT	
Diagonal D1 (North)	Lattice Pl. (Bottom)	9.7				Reference UT	
Post V1 (North)	Web	10.2				Reference UT	
End Post D0 (South)	Web (North)	9.9				Reference UT	
	Top Flange	9.4				Reference UT	
End Post D0 (North)	Web (South)	9.6				Reference UT	
	Top Flange	10				Reference UT	
Stringer S2 at BC7	Web (North)	11.5	8.0	8.1	8.5	Corrosion 700mm length adjacent to bottom flange	1
Stringer S3 at BC7	Web (North)	11.5	8.8			200mm length adjacent to bottom flange	2
			8.0			100mm length adjacent to bottom flange	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET SWING BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: 45° ☐ 60° ☐ 70° ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Stringer S5 at L8 (West)	Web (South)	11.6	7.8	100mm length adjacent to bottom flange	3
L8 / FB8 at S2 (North)	Web (West)	12.6	9.2	100mm length adjacent to bottom flange	4
	Bottom Flange (West)	15.6	12.2	100mm length	
Girder B (South)	Bottom Flange (Angle)	10.3	4.6	200mm length	5
Hub Floor Brace FB4 (West)	Bottom Flange (Angle)	11.2	8.6	500mm length corrosion	6-1 & 6-2
Floor Brace FB0 (East) at Stringer S3 (South)	Web	13.2	10.2	Corrosion at bottom support angle	7-1 & 7-2
Floor Brace FB0 (East) at Stringer S4 (North)	Web	12.6	4.6	Corrosion at bottom support angle >50%	8
Floor Brace FB0 (East) at Stringer S2 (South)	Web	12.8	10.2	Corrosion at top connector angle	9
		12.8	10.8	Corrosion at bottom connector angle	10

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

A. Aitken

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET SWING BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: 45° ☐ 60° ☐ 70° ☐ OTHER ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Floor Brace FB1 (West) at Stringer S2 (South)	Web	13.0	9.0	Corrosion at bottom support angle	11
Floor Brace FB1 (West) at Stringer S2 (South)	Web	12.9	6.6	Corrosion at bottom of web connector angle	12
Floor Brace FB2 at Stringer S0 (North)	Web	12.5	9.5	Corrosion at bottom support angle	13
Stringer S1 (North) at BC3 quadrant	Web	11.0	8.5-9.3	>10% surface corrosion 600mm length	14
Floor Brace FB3 (East) at Stringer S2 (South)	Web	12.4	5.5	Corrosion front and back surface (pit gauged)	15

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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MAGNETIC PARTICLE REPORT

C.B. FILE: 11-09-69-MT

CUSTOMER: DELCAN (OTTAWA)
P.O. #: B02211 BOB **CONT.#:** PCA.PROJ.NO. 2011-4650-20027340
PART NAME: HAMLET SWING BRIDGE COMPONENTS
PART/DWG #: BRIDGE #57 (TRENT SEVERN WATER WAY
MATL.TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: OCTOBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CAN/CGSB 48.9712 LEVEL: 2
SNT-TC-1A LEVEL: II

METHOD: HEAD ☐ **COIL** ☐ **C/CON** ☐ **AMPS**
AC ☐ **DC** ☒ **PRODS** ☐ **YOKE** ☒ **SPACING** 4"-6"
INSTRUMENT: PERMANENT MAGNETS **S/N:** TCM 50
BLACK LIGHT: ☐ N/A **S/N:**
WHITE LIGHT: ☒ FLASHLIGHT
PARTICLE METHOD: DRY CONTINUOUS
TYPE OF MEDIUM: MAGNAFLUX NO. 2 YELLOW
PROCEDURE: QCP MT.3.2 **REV.** 1
SPECIFICATION: CSA W59-03M
ACCEPTANCE CRITERIA: CHECK FOR CRACKS

THIS REPORT REFERS TO THE MAGNETIC PARTICLE EXAMINATION OF THE FOLLOWING:

ITEM:	AREA INSPECTED:	DISPOSITION:
STRINGER S5 (SOUTH) AT L9 (WEST)	FLAME CUT, ROUGH EDGED CUT AWAY NOTCH AT TOP FLANGE / WEB.	ACCEPTABLE
STRINGER S3 (SOUTH) AT L8 (EAST)	FLAME CUT, ROUGH EDGED CUT AWAY NOTCH AT TOP FLANGE / WEB.	ACCEPTABLE
STRINGER S2 (NORTH) AT L8 (WEST)	FLAME CUT, ROUGH EDGED CUT AWAY NOTCH AT TOP FLANGE / WEB.	ACCEPTABLE
STRINGER S5 (NORTH) AT L8 (EAST)	FLAME CUT, ROUGH EDGED CUT AWAY NOTCH AT TOP FLANGE / WEB.	ACCEPTABLE
STRINGER S3 (SOUTH) AT FB2	FLAME CUT, ROUGH EDGED CUT AWAY NOTCH AT TOP FLANGE / WEB.	ACCEPTABLE

AT THE TIME OF OUR INSPECTION THE ABOVE LISTED ITEMS WERE FOUND TO BE ACCEPTABLE TO THE ABOVE MENTIONED SPECIFICATION / CRITERIA.

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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VISUAL INSPECTION REPORT

C.B. FILE: 11-09-69-VT

CUSTOMER: DELCAN (OTTAWA)
P.O. #: B02211 BOB **BRIDGE #:** 57
PART / DWG.#: PCA PROJECT #: 2011-4650-20027340
PROJECT: HAMLET, ONTARIO SWING BRIDGE #57
JOB LOCATION: HAMLET, ONTARIO (TRENT SEVERN WATERWAY)
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011

INSPECTOR: DAVID GUEST
CWB/CSA W178.2 LEVEL: 2 **AWS QC1:**
SPECIFICATION: CSA W59-03
ACCEPTANCE CRITERIA: CL. 12, AS PER CLIENT REQUEST
REPORT #: 1 **INSPECTION: SHOP** ☐ **FIELD** ☒
IN FIELD: WEATHER: N/A **TEMP:**

ITEM #:	MEMBER	DESCRIPTION	REMARKS	Photo #:
Top Chord TC7 (North) at U8, V8 (Top)	Paint Coating	Loss of coating protection	Surface corrosion <10%	
Post V5, V4 Railing Lattice (South)	Paint Coating	Loss of coating protection	Surface corrosion <10%	
Center Diagonal (South)	V4 to V5 angle	Top leg of angle “delaminated”	>90% length of member	16-1 & 16-2
Stringer S5 at L9 (West)	Web / Top Flange	Flame cut rough edges in top flange / web	‘Sharp’ non-radius’d (stress riser)	
Stringer S3 at L8 (East)	Web / Top Flange	Flame cut rough edges in top flange / web	‘Sharp’ non-radius’d (stress riser)	
Stringer S2 at L8 (West)	Web / Top Flange	Flame cut rough edges in top flange / web	‘Sharp’ non-radius’d (stress riser)	
Stringer S5 at L8 (East)	Web / Top Flange	Flame cut rough edges in top flange / web	‘Sharp’ non-radius’d (stress riser)	
L8 / FB8 (West) at S2 (North)	Web Coating	Loss of coating protection		
Cross Bracing at S3 and BC5 quadrant	Angles	Through corrosion / perforation	200mm length through corrosion	17

SUBMITTED BY: ALASTAIR AITKEN

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VISUAL INSPECTION REPORT

C.B. FILE: 11-09-69-VT

CUSTOMER: DELCAN (OTTAWA)
P.O. #: B02211 BOB **BRIDGE #:** 57
PART / DWG.#: PCA PROJECT #: 2011-4650-20027340
PROJECT: HAMLET, ONTARIO SWING BRIDGE #57
JOB LOCATION: HAMLET, ONTARIO (TRENT SEVERN WATERWAY)
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011

INSPECTOR: DAVID GUEST
CWB/CSA W178.2 LEVEL: 2 **AWS QC1:**
SPECIFICATION: CSA W59-03
ACCEPTANCE CRITERIA: CL. 12, AS PER CLIENT REQUEST
REPORT #: 1 **INSPECTION: SHOP** ☐ **FIELD** ☒
IN FIELD: WEATHER: N/A **TEMP:**

ITEM #:	MEMBER	DESCRIPTION	REMARKS	Photo #:
Cross Bracing at S3 and BC6 quadrant	Angle	Through corrosion / perforation	20mm length	18
Hub	Paint / coating Girder B (West)	Loss / deterioration	Corrosion >10%	19
Hub at Girder 'C' (North & South)	Top Flange angle support	Total angle loss / deterioration	Corrosion 100%	20
Hub Girder 'A' (West)	Connector Plate(s) at X-Brace and bottom flange	Corrosion 'knife-dge' of connector plate top Corrosion .50% bottom flange and connector plate		21-1, 21-2 & 21-3

SUBMITTED BY: ALASTAIR AITKEN

CANADIAN WELDING BUREAU 519 A. AITKEN LEVEL III CSA W178.2 CERTIFIED WELDING

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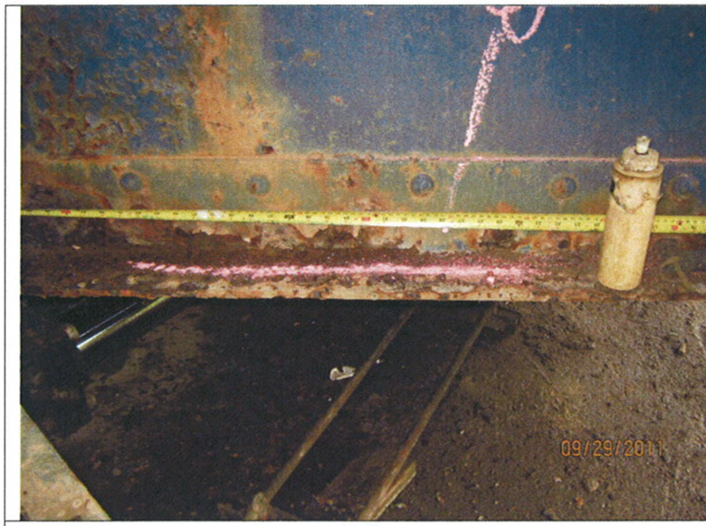

09/29/2011

6-2

W.ABUT
FBO
REF 13.2
S2
10.2
09/29/2011

7-1

W.ABUT
FBO
REF 13.2
S2
10.2
09/29/2011

7-2

12.6 REF
4.6
09/29/2011

8

FBO
W.ABUT
REF 12.8
10.2
10.8
WEST FACE OF FBO
S2
09/29/2011

9

S2
S
09/29/2011

10

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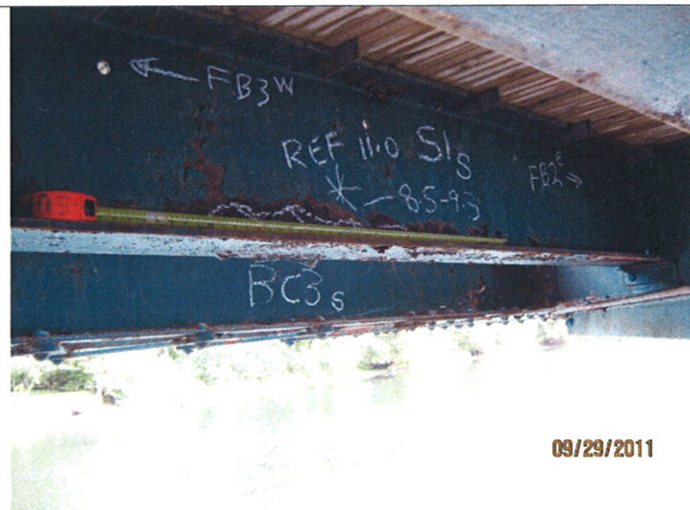

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October 17th, 2011

Delcan Corporation
1223 Michael Street
Suite 100
Ottawa, Ontario
K1J 7T2

Attention: Mr. Peter Harvey B.A. Sc, E.I.T.

Our File: 11-09-69-UT

Subject: Hamlet, Ontario Fixed Bridge #57
Parks Canada Agency Project #: 2011-4650-20027340

1.0 Scope:

1.1 This report covers the structural steel corrosion survey inspection of the above noted fixed bridge structure, located in Hamlet, Ontario.
The inspection consisted of visual inspection, ultrasonic thickness testing of representative members where deemed necessary / as directed by client. Testing of members was limited due to time constraints and unprepared surfaces.
Inspection date: September 28th & 29th, 2011

2.0 Observations:

Please note: The items listed herein are references to the attached inspection report pages and digital photographs as applicable.

2.1 Cross beam (flange & web sections) exhibited severe corrosion / section loss as noted on ultrasonic report and photo #'s 1, 3, 5, 6, 7 & 8.

2.2 Vertical truss posts (flanges) at pinned connections exhibited severe corrosion per ultrasonic report and photo #'s 2 & 4. Also see visual report and photo # 9.

2.3 Bottom chord tension member(s) / forged eye (ends) exhibited severe corrosion at the East abutment pinned connection points as per visual report and photo #'s 10 & 11.

2.4 1 vertical truss post exhibited mechanical damage (previous impact) with deflection noted on visual inspection report and photo # 12.

2.5 Thickness readings obtained were isolated to areas exhibiting greater than 10% (visual) loss. Access and time constraints limited the number of readings obtained. All accessible corrosion areas were recorded for client review / evaluation.

3.0 Results:

3.1 The attached ultrasonic thickness readings, visual inspection reports and magnetic particle inspection reports pages 1 through 7 inclusive conclude this report.

Should you have any further questions regarding this report, please do not hesitate to contact this office.

Inspected by: David Guest C.E.T.

Submitted by: A. Aitken

Alastair Aitken
CSA W178.2 Level III

CANADIAN WELDING
519
A. AITKEN
LEVEL III
CSA W178
CERTIFIED WELDING

Q.A. Manager
C.B. Non-Destructive Testing Ltd.

Attachments: Inspection reports (7 pages)

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET FIXED BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Cross Beam L6	Bottom Flange	13.7	8.8	Flange toe to web corrosion (80mm long)	1-1 & 1-2
Cross Beam L6	Web	10.0		Reference UT	
Post V5 (North)	Flange (South-East)	10.0	6.4	Corrosion at pinned connection	2
Cross Beam L5	Bottom Flange	13.4	8.9	Flange toe to web corrosion	3
Stringer S5 & S6	Web	5.4		Reference UT	
(North-West)	Bottom Flange	4.6		Reference UT	
Post V4 North	Flange (South-East)	10.4	3.2	Corrosion at pinned connection	4
Cross Beam L4 (East) at S7	Web	10.7	5.8	Corrosion adjacent to flange	5
Cross Beam L4 (East) at S3	Web	10-10.7	6.2-8.3	Corrosion adjacent to flange	6

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET FIXED BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Cross Beam L3 (West) at S3 & S4	Web	10.5	4.4-6.2	Corrosion adjacent to flange	7
Cross Beam L2 (East) at S7	Top Flange	13.7-14.7	6.9-8.0	Corrosion at S7 bottom flange	8
Top Chord TC5 (South) at V5	Top Flange	8.1-8.5		Reference UT	
Strut U5 (South)	Flange	6.6-7.1		Reference UT	
Top Chord TC3 (North) at U4	Top Flange	8.6-9.0		Reference UT	
	Web	9.2-9.5		Reference UT	
Post V4 (North)	Flange	12.7-13.0		Reference UT	
Top Chord TC1 (North) at U2	Top Flange	8.2-8.6		Reference UT	
	Web	8.9-9.4		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

A Aitken

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET FIXED BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Post V2 (North) AT U2	Flange (South West)	10.8-11.0		Reference UT	
Top Chord TC1 at U1	Top Flange	9.1		Reference UT	
Post V3 (South)	Flange (North East)	11.2	10.4	Corrosion at section 'B' brace connector plate & vertical truss V3 is <10%	
Top Chord TC5 (South)	Top Flange	8.8		Reference UT	
	Conn. Pl.	7.4		Reference UT	
Post V6 (North)	Web	5.8		Reference UT	
(South)	Web	5.6		Reference UT	
Post V5 (North)	Web	8.7		Reference UT	
(South)	Web	8.8		Reference UT	
Post V4 (North)	Web	9.3		Reference UT	
Post V3 (South)	Web	9.6		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET FIXED BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: STRESSTEL T-MIKE EM+
SERIAL #: TCU 22
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.4.1 **REV.** 1
SPECIFICATION: ASTM E797-05
ACCEPTANCE CRITERIA: RECORD THICKNESS

TRANSDUCER MAKE: STRESSTEL

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐ DUAL

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: 45° ☐ 60° ☐ 70° ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

Item	Member	Reference / Adjacent 'T' mm	Thickness Reading mm: 1	Remarks:	Photo #:
Post V2 (North)	Web	9.9		Reference UT	
Post V1 (South)	Web	6.1		Reference UT	
End Post D0 (North)	Top Flange	8.1		Reference UT	
	Web (South)	8.1		Reference UT	
End Post D0 (South)	Top Flange	8.0		Reference UT	
	Web (North)	8.7		Reference UT	
End Post D6 (North)	Top Flange	8.1		Reference UT	
	Web (South)	9.0		Reference UT	
End Post D6 (South)	Top Flange	7.6		Reference UT	
	Web (North)	8.2		Reference UT	

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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ULTRASONIC REPORT

C.B. FILE: 11-09-69-UT

CUSTOMER: DELCAN (OTTAWA)
P.O.#: B02211 BOB **SITE #:** BRIDGE # 57
BRIDGE NAME: HAMLET FIXED BRIDGE
PART/DWG #.: PCA PROJECT #: 2011-4650-20027340
MATL. TYPE: STEEL **THK:** VARIOUS
JOB LOCATION: HAMLET, ONTARIO
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011
TECHNICIAN: DAVID GUEST
CGSB 48.9712 LEVEL: 1 **SNT-TC-1A LEVEL:** I

TECHNIQUE: PULSE ECHO CONTACT
SURFACE FINISH: PAINTED / RUSTED
COUPLANT: ECHOGEL 20
INSTRUMENT: USN 52
SERIAL #: TCU 03
CALIBRATION STD.: STEP BLOCKS
PROCEDURE #: UT.2.2 **REV.** 4
SPECIFICATION: ASTM A388-10
ACCEPTANCE CRITERIA: CHECK FOR CRACKING

TRANSDUCER MAKE: GE BENCHMARK

Normal Beam

1/2" Dia ☐ **1" Dia** ☐ **OTHER** ☒ 0.250" DIA.
2.25 Mhz ☐ **5 Mhz** ☒ **OTHER** ☐

Angle Beam

1/2" Dia. ☐ **1/2" Sq** ☐ **OTHER** ☐
2.25 Mhz ☐ **5 Mhz** ☐ **OTHER** ☐
Shoe Angle: **45°** ☐ **60°** ☐ **70°** ☐ **OTHER** ☐

THIS REPORT REFERS TO THE ULTRASONIC EXAMINATION OF THE FOLLOWING:

ALL ACCESSIBLE BOLTS AND PINS ON THE FIXED (LIGHT TRUSS DESIGN) BRIDGE WERE INSPECTED FROM THE BOLT POINT END / ACCESSIBLE PIN END.

NO EVIDENCE OF CRACKED BOLTS WERE FOUND, THUS CONSIDERED ACCEPTABLE.

NO EVIDENCE OF CRACKED PINS WERE FOUND, THUS CONSIDERED ACCEPTABLE.

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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MAGNETIC PARTICLE REPORT

C.B. FILE: 11-09-69-MT

CUSTOMER: DELCAN (OTTAWA)	**METHOD: HEAD** ☐ **COIL** ☐ **C/CON** ☐ **AMPS**
P.O. #: B02211 BOB **CONT.#:** PCA.PROJ.NO. 2011-4650-20027340	**AC** ☐ **DC** ☒ **PRODS** ☐ **YOKE** ☒ **SPACING** 4"-6"
PART NAME: HAMLET FIXED BRIDGE COMPONENTS	**INSTRUMENT:** PERMANENT MAGNETS **S/N:** TCM 50
PART/DWG #: BRIDGE #57 (TRENT SEVERN WATER WAY	**BLACK LIGHT:** ☐ N/A **S/N:**
MATL.TYPE: STEEL **THK:** VARIOUS	**WHITE LIGHT:** ☒ FLASHLIGHT
JOB LOCATION: HAMLET, ONTARIO	**PARTICLE METHOD:** DRY CONTINUOUS
INSPECTION DATE: OCTOBER 28TH & 29TH, 2011	**TYPE OF MEDIUM:** MAGNAFLUX NO. 2 YELLOW
TECHNICIAN: DAVID GUEST	**PROCEDURE:** QCP MT.3.2 **REV.** 1
CAN/CGSB 48.9712 LEVEL: 2	**SPECIFICATION:** CSA W59-03M
SNT-TC-1A LEVEL: II	**ACCEPTANCE CRITERIA:** CHECK FOR CRACKS

THIS REPORT REFERS TO THE MAGNETIC PARTICLE EXAMINATION OF THE FOLLOWING:

ITEM:	AREA INSPECTED:	DISPOSITION:
HANGER ROD'S (1" SQ.)	TOP / BENT END'S AT L1 THROUGH L6 (NORTH & SOUTH).	ACCEPTABLE
BOTTOM CHORD END'S / EYE'S OF BC0 TO BC6 INCLUSIVE	CURVED END'S OF EYE'S (ACCESSIBLE) AT PINNED LOCATIONS L1 THROUGH L5 (NORTH & SOUTH).	ACCEPTABLE
DIAGONAL END / EYE'S OF D1 THOUGH D4 INCLUSIVE	CURVED END'S OF EYE'S (ACCESSIBLE) AT PINNED LOCATIONS L2 THROUGH L5 (NORTH & SOUTH).	ACCEPTABLE
DIAGONAL BRACE 'D1' AT V1 (NORTH) / U1 ELEVATION	EYE (FORGED JOINT) TO BAR LENGTH OF DIAGONAL.	ACCEPTABLE

AT THE TIME OF OUR INSPECTION THE ABOVE LISTED ITEMS WERE FOUND TO BE ACCEPTABLE TO THE ABOVE MENTIONED SPECIFICATION / CRITERIA.

REMARKS:

SUBMITTED BY: ALASTAIR AITKEN

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VISUAL INSPECTION REPORT

C.B. FILE: 11-09-69-VT

CUSTOMER: DELCAN (OTTAWA)
P.O. #: B02211 BOB **BRIDGE #:** 57
PART / DWG.#: PCA PROJECT #: 2011-4650-20027340
PROJECT: HAMLET, ONTARIO FIXED BRIDGE #57
JOB LOCATION: HAMLET, ONTARIO (TRENT SEVERN WATERWAY)
INSPECTION DATE: SEPTEMBER 28TH & 29TH, 2011

INSPECTOR: DAVID GUEST
CWB/CSA W178.2 LEVEL: 2 **AWS QC1:**
SPECIFICATION: CSA W59-03
ACCEPTANCE CRITERIA: CL. 12, AS PER CLIENT REQUEST
REPORT #: 1 **INSPECTION: SHOP** ☐ **FIELD** ☒
IN FIELD: WEATHER: N/A **TEMP:**

ITEM #:	MEMBER	DESCRIPTION	REMARKS	Photo #:
V2 Post (South) at L2	Flange (North West)	Flange Corrosion <50% section loss to knife-edge, Flange to pin sleeve		9
V6 End Post (North) above L6 pinned connection	I Beam	Deflection off of vertical	From previous bridge impact (vehicular)	10-1 & 10-2
Cross Beam L5 (South) at BC5	Web, Flange	Paint deterioration (typical condition of bridge)	Surface corrosion <10%	
Strut U5 (South)	Plate, Flange	Paint deterioration (typical condition of bridge)	Surface corrosion <10%	
Bottom Chord BC6 at L7 (North)	Forged Eye Ends (Bottom)	*>90% section loss / corrosion*	*Tension Member*	11
Bottom Chord BC6 at L7 (South)	Forged Eye Ends (Bottom)	*>90% section loss / corrosion*	*Tension Member*	12-1 & 12-2

SUBMITTED BY: ALASTAIR AITKEN

CANADIAN WELDING BUREAU 519 A. AITKEN LEVEL III CSA W178.2 CERTIFIED WELDING INSPECTOR

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1-1

1-2

2

3

4

5

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6

7

8

9

10-1

10-2

11

12-1

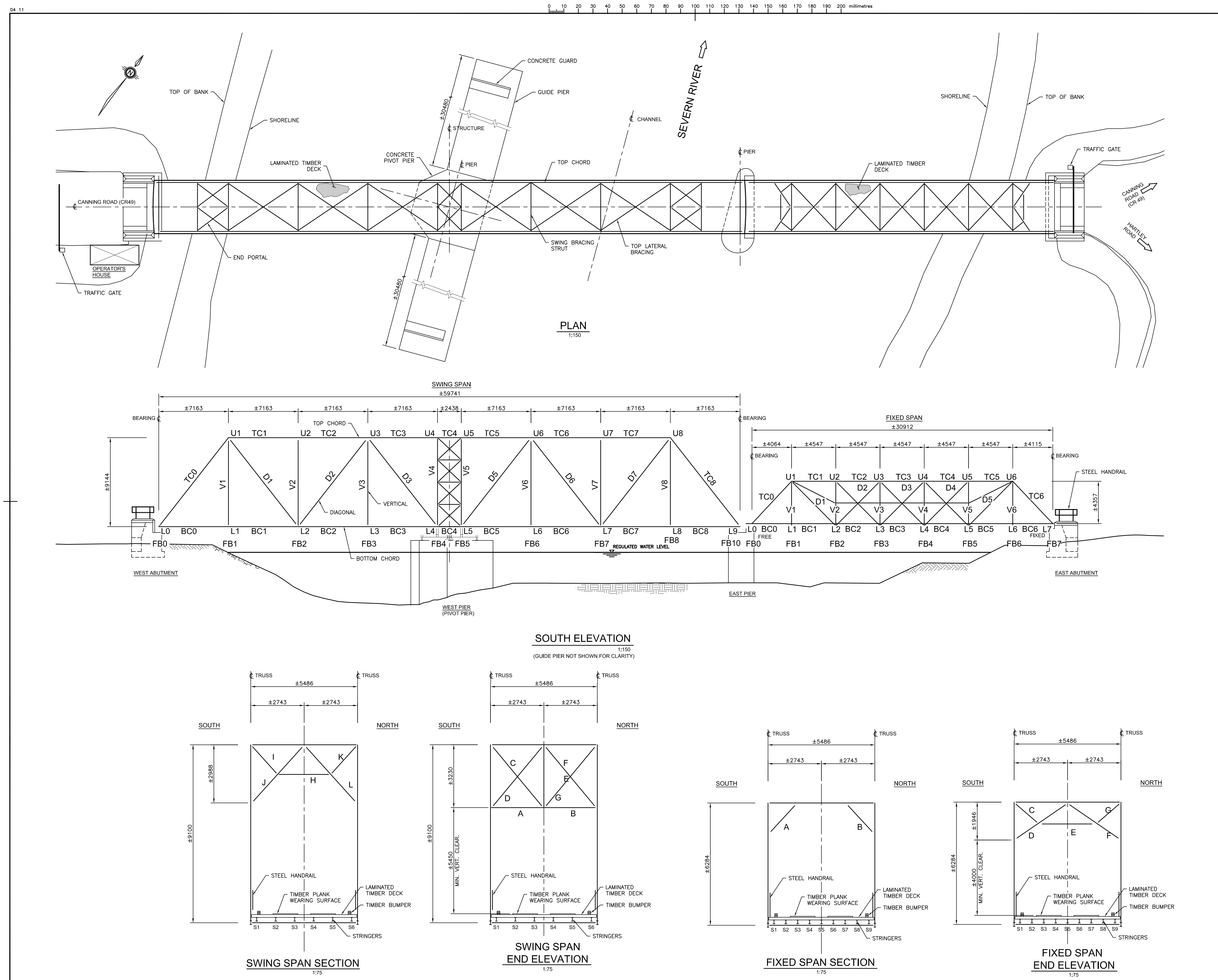
12-2

APPENDIX D

UNDERWATER INSPECTION VIDEO OF PIERS (DVD)

APPENDIX E

DRAWINGS



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Delcan

revision		date
04		
03		
02		
01		

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

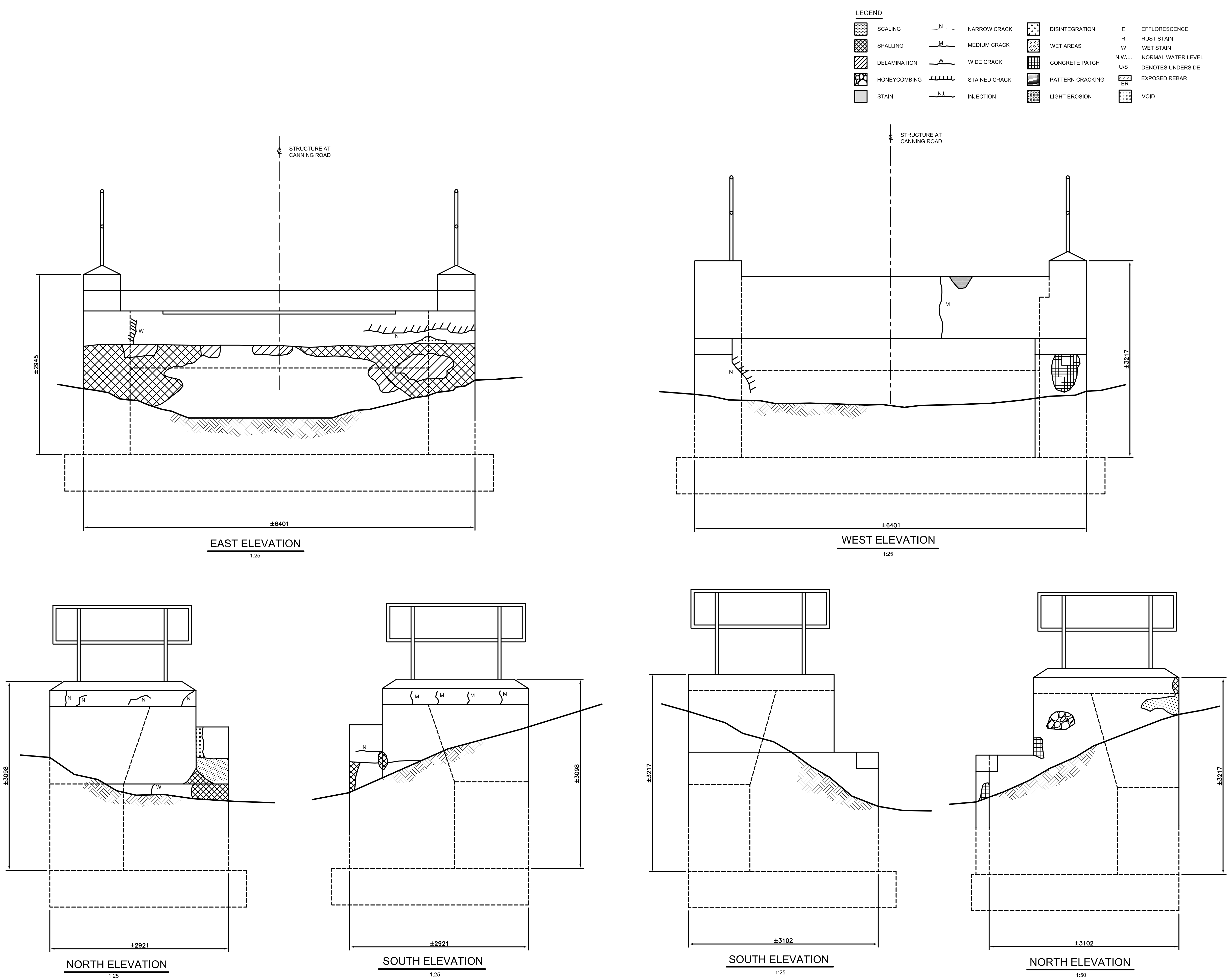
A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

drawing title / titre du dessin
GENERAL ARRANGEMENT

drawn by / dessiné par: RD
designed by / conçu par: ----
approved by / approuvé par: PM
tender / soumission: DUNCAN MANSER — project manager / administrateur de projets
project date / date du projet: MARCH 2012
project no. / no. du projet: 2011-4650-20027340
drawing no. / dessin no.: B-1

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



EAST ELEVATION
1:25

WEST ELEVATION
1:25

NORTH ELEVATION
1:25

SOUTH ELEVATION
1:25

SOUTH ELEVATION
1:25

NORTH ELEVATION
1:50

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revision		date

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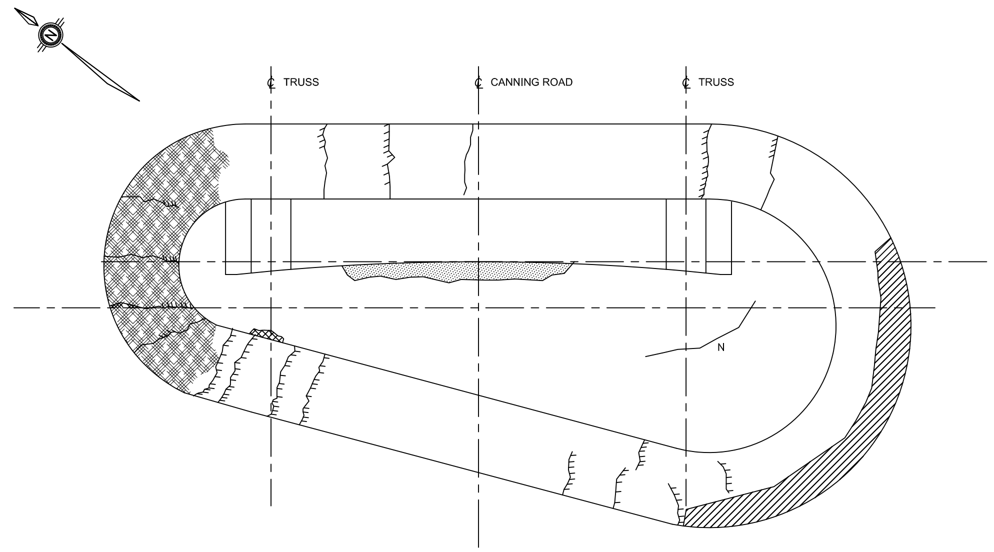
A Detail No. / No. du détail
B drawing no. – where detail required / dessin no. – où détail exigé
C drawing no. – where detailed / dessin no. – où détaillé

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

drawing title / titre du dessin
ABUTMENT DETERIORATION

drawn by / dessiné par: RD
designed by / conçu par: ____
approved by / approuvé par: PM
tender / soumission: DUNCAN MANSER — project manager / administrateur de projets
project date / date du projet: MARCH 2012
project no. / no. du projet: 2011-4650-20027340
drawing no. / dessin no.: B-2

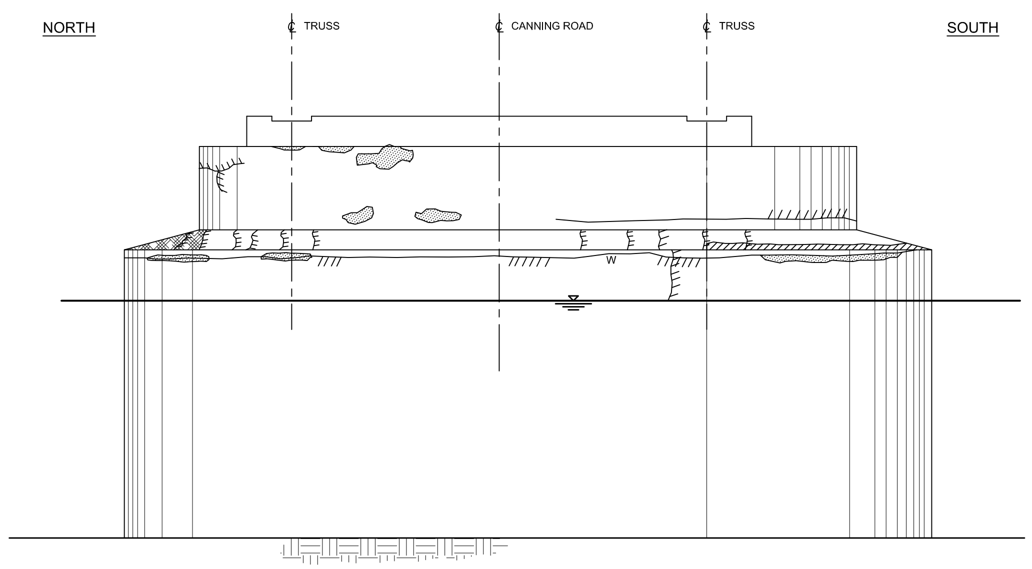
GSC-B1 DATE PLOTTED: PLOT SCALE: CAD FILE:



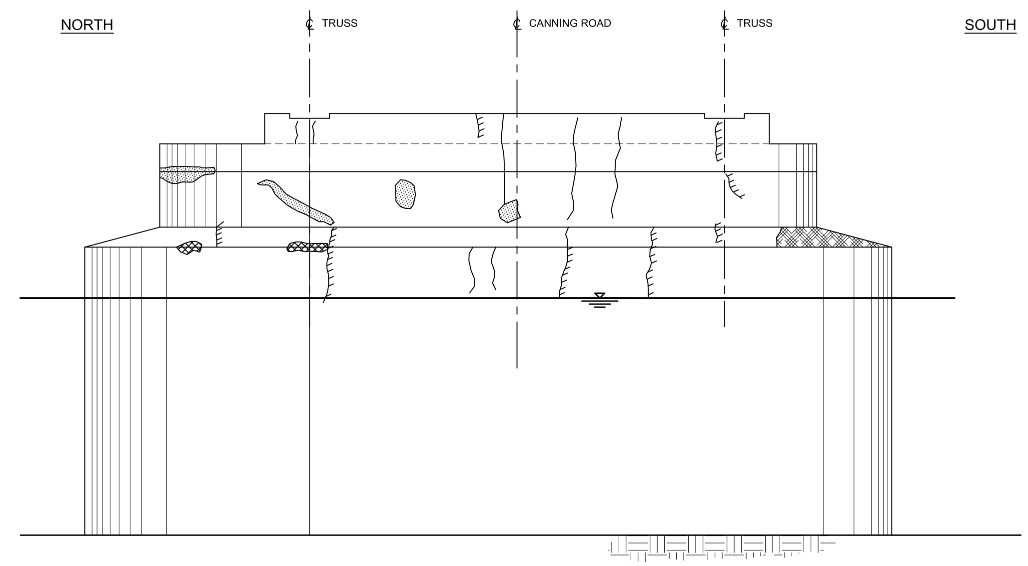
PIER PLAN
1:50

LEGEND

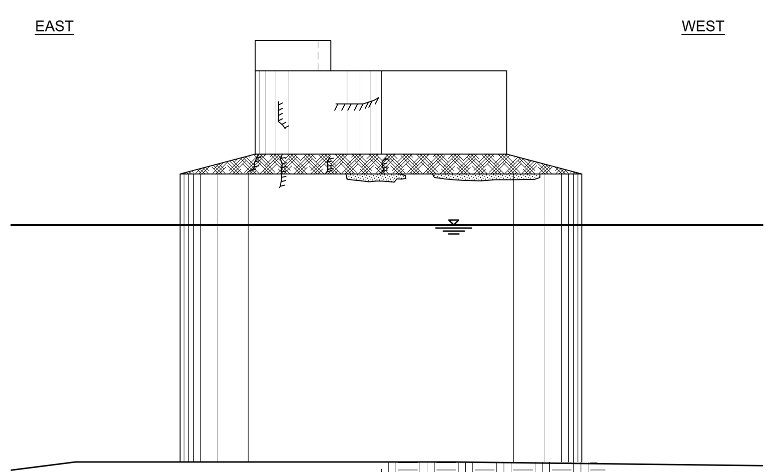
	SCALING	N	NARROW CRACK		DISINTEGRATION	E	EFFLORESCENCE		
	SPALLING	M	MEDIUM CRACK		WET AREAS	R	RUST STAIN		
	DELAMINATION	W	WIDE CRACK		CONCRETE PATCH	W	WET STAIN		
	HONEYCOMBING		STAINED CRACK		PATTERN CRACKING	N.W.L.	NORMAL WATER LEVEL		
	STAIN	INJ.	INJECTION		LIGHT EROSION	U/S	DENOTES UNDERSIDE		
						ER	EXPOSED REBAR		
							VOID		



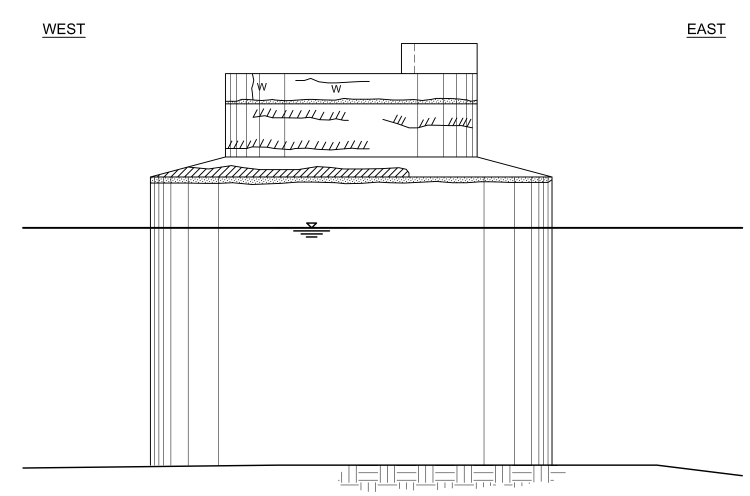
WEST ELEVATION
1:40



EAST ELEVATION
1:40



NORTH ELEVATION
1:40



SOUTH ELEVATION
1:40

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revision		date

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

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

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

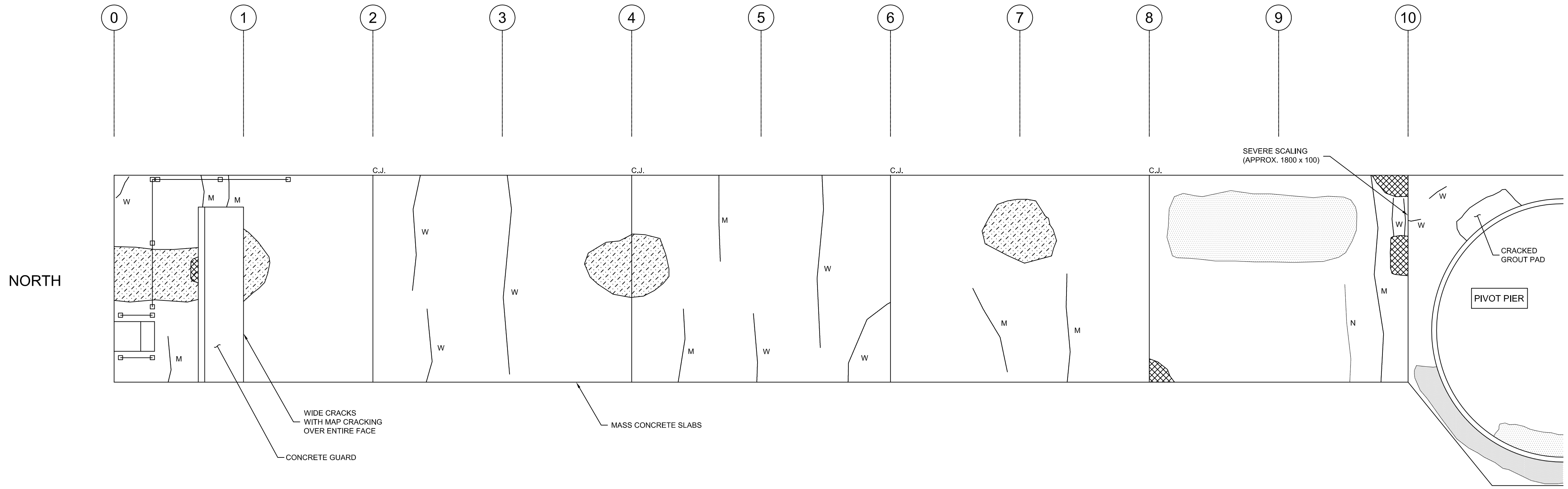
drawing title / titre du dessin
EAST PIER DETERIORATION

drawn by / dessiné par: RD
designed by / conç par: ----
approved by / approuvé par: PM
tender / soumission: DUNCAN MANSER — project manager / administrateur de projets
project date / date du projet: MARCH 2012
project no. / no. du projet: 2011-4650-20027340
drawing no. / dessin no.: B-3

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

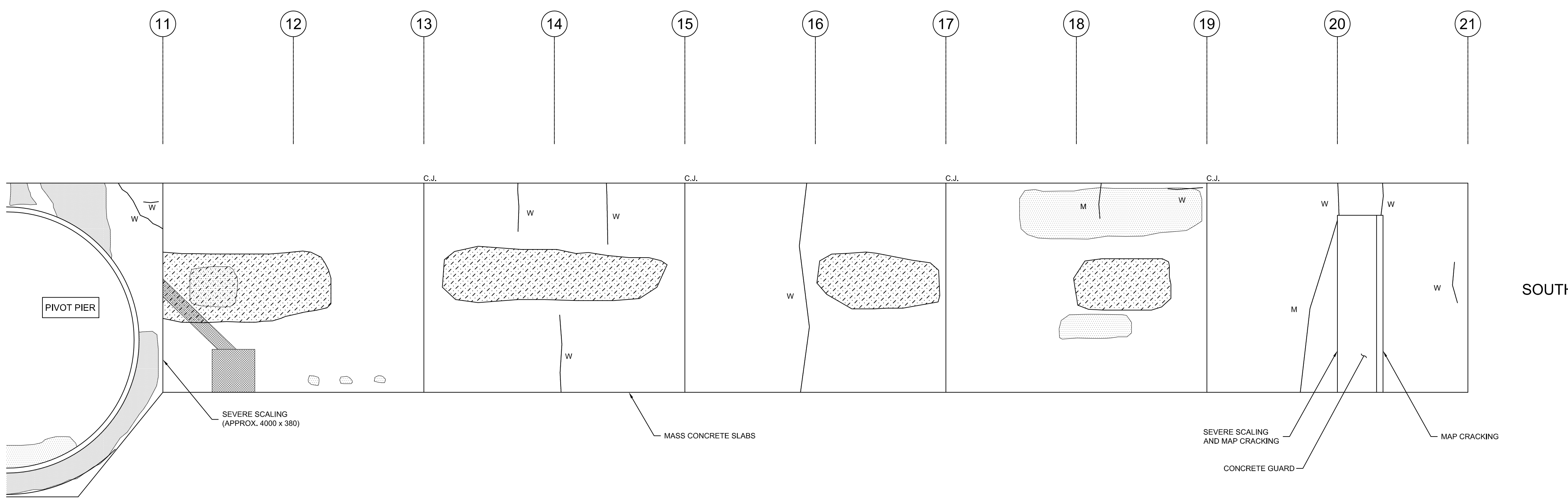
LEGEND

LIGHT SCALING	N — NARROW CRACK	MEDIUM SCALING	E EFFLORESCENCE
SPALLING	M — MEDIUM CRACK	WET AREAS	R RUST STAIN
DELAMINATION	W — WIDE CRACK	CONCRETE PATCH	W WET STAIN
HONEYCOMBING	STAINED CRACK	PATTERN CRACKING	N.W.L. NORMAL WATER LEVEL
STAIN	INJ. — INJECTION	LIGHT EROSION	U/S DENOTES UNDERSIDE
			ER EXPOSED REBAR
			VOID



TOP OF CRIB - NORTH PLAN

1:50



TOP OF CRIB - SOUTH PLAN

1:50

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

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

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

drawing title / titre du dessin
WEST PIER DETERIORATION I (GUIDE PIER)

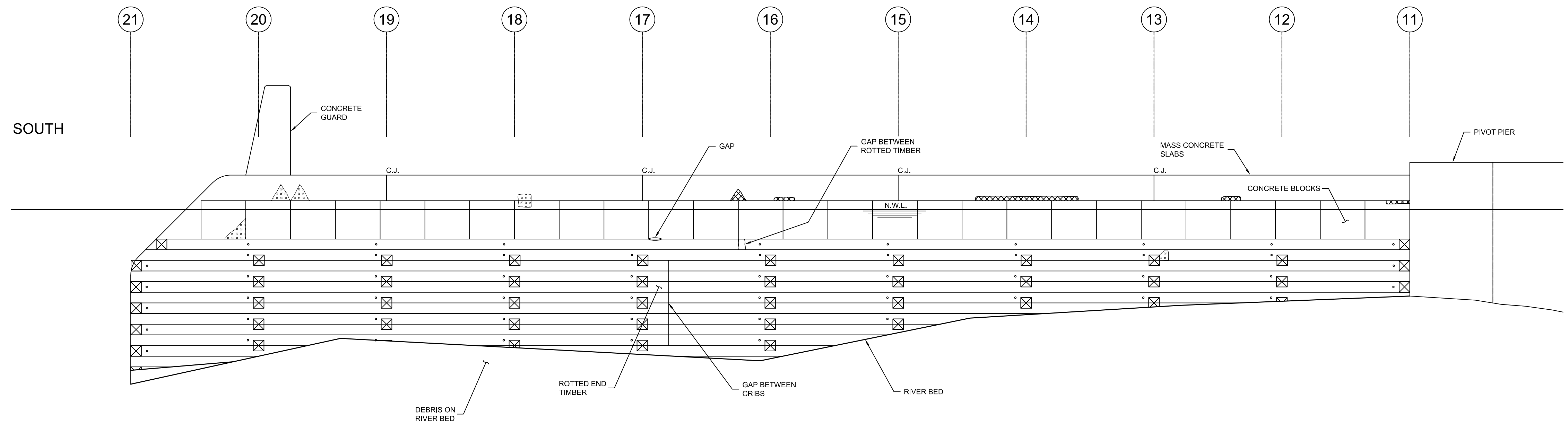
drawn by / dessiné par: SAS
designed by / conçu par: ----
approved by / approuvé par: PM
tender / soumission: DUNCAN MANSER — project manager / administrateur de projets
project date / date du projet: MARCH 2012
project no. / no. du projet: 2011-4650-20027340
drawing no. / dessin no.: B-4

04 11

0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

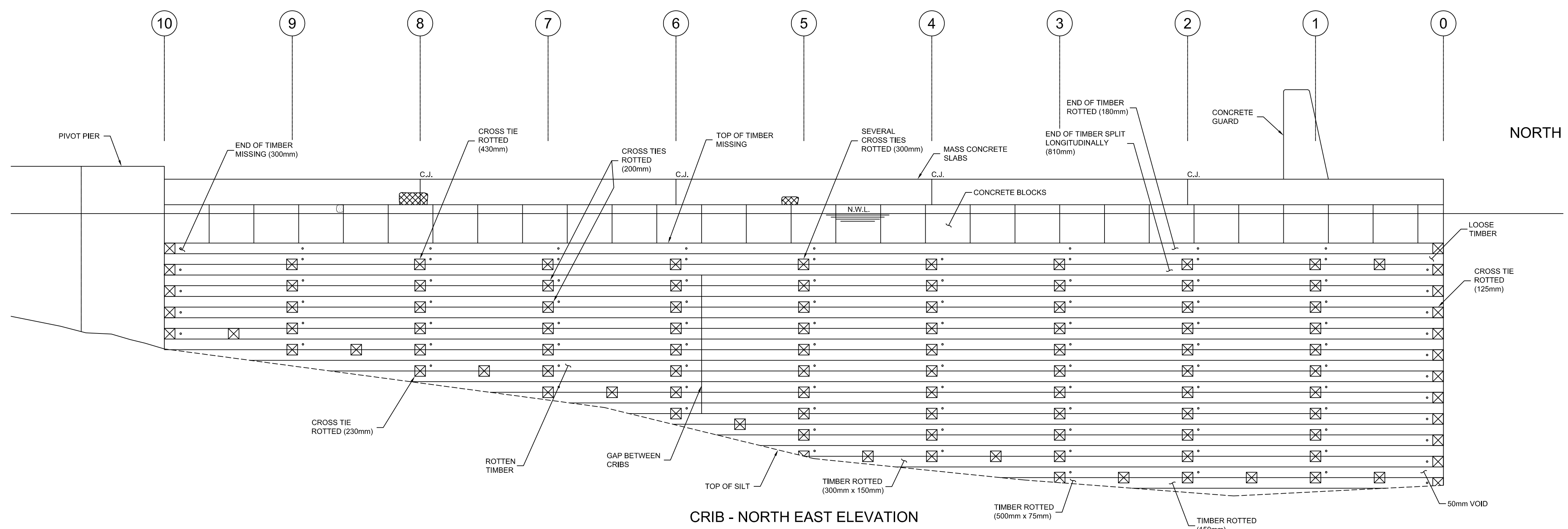
LEGEND

LIGHT SCALING	N NARROW CRACK	MEDIUM SCALING	E EFFLORESCENCE
SPALLING	M MEDIUM CRACK	WET AREAS	R RUST STAIN
DELAMINATION	W WIDE CRACK	CONCRETE PATCH	W WET STAIN
HONEYCOMBING	STAINED CRACK	PATTERN CRACKING	N.W.L. NORMAL WATER LEVEL
STAIN	INJ. INJECTION	LIGHT EROSION	U/S DENOTES UNDERSIDE
			ER EXPOSED REBAR
			VOID



CRIB - SOUTH EAST ELEVATION

1:50



CRIB - NORTH EAST ELEVATION

1:50

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Delcan

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

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

project title / titre du projet

HAMLET BRIDGE (BRIDGE 57)

COMPREHENSIVE DETAILED INSPECTION

HAMLET ONTARIO

drawing title / titre du dessin

WEST PIER DETERIORATION II (GUIDE PIER)

drawn by / dessine par: SAS

designed by / conc par: ----

approved by / approuve par: PM

tender / soumission: DUNCAN MANSER — project manager / administrateur de projets

project date / date du projet: MARCH 2012

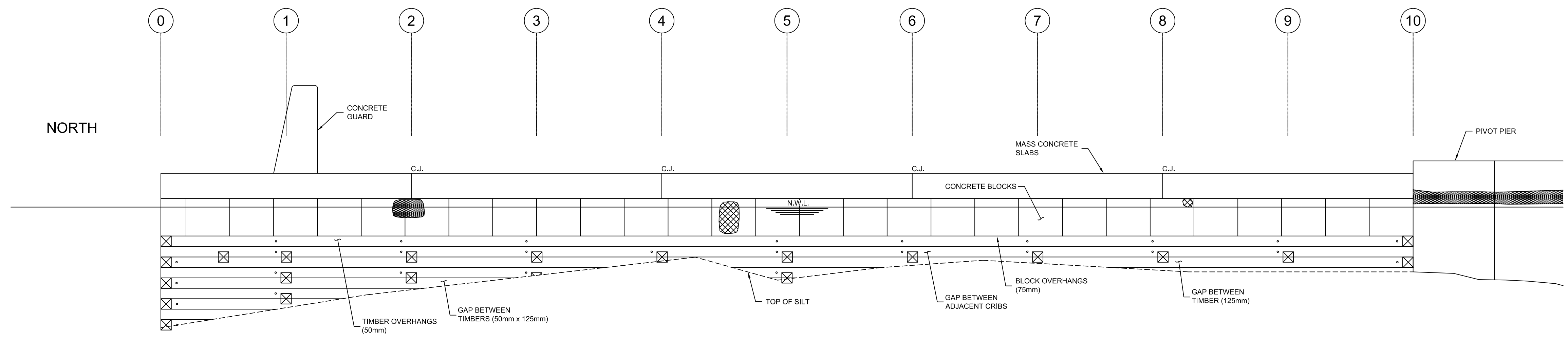
project no. / no. du projet: **2011-4650-20027340**

drawing no. / dessine no.: **B-5**

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

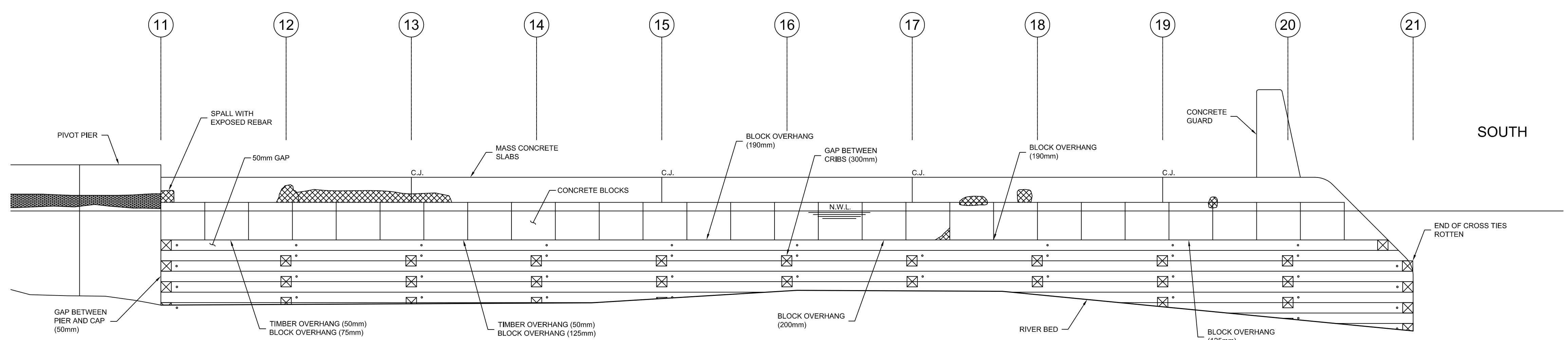
LEGEND

	LIGHT SCALING	N	NARROW CRACK		MEDIUM SCALING	E	EFFLORESCENCE
	SPALLING	M	MEDIUM CRACK		WET AREAS	R	RUST STAIN
	DELAMINATION	W	WIDE CRACK		CONCRETE PATCH	W	WET STAIN
	HONEYCOMBING		STAINED CRACK		PATTERN CRACKING	N.W.L.	NORMAL WATER LEVEL
	STAIN	INJ.	INJECTION		LIGHT EROSION	U/S	DENOTES UNDERSIDE
						ER	EXPOSED REBAR
							VOID



CRIB - SOUTH WEST ELEVATION

1:50



CRIB - NORTH WEST ELEVATION

1:50

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

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

Parks Canada Parcs Canada Canada

Delcan

revision		date
04		
03		
02		
01		

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

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

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

drawing title / titre du dessin
WEST PIER DETERIORATION III (GUIDE PIER)

drawn by / dessiné par: SAS

designed by / conc par: ----

approved by / approuvé par: PM

tender / soumission: DUNCAN MANSER — project manager / administrateur de projets

project date / date du projet: MARCH 2012

project no. / no. du projet: 2011-4650-20027340

drawing no. / dessin no.: B-6

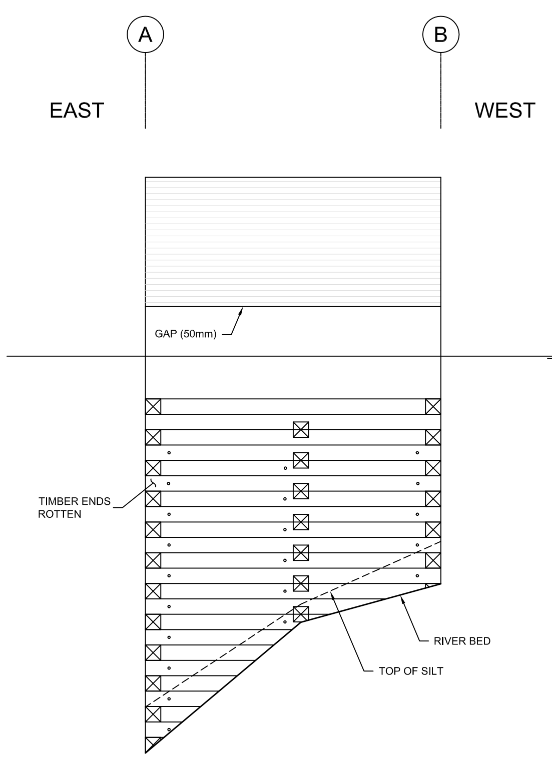
GSC-B1 DATE PLOTTED: PLOT SCALE: CAD FILE:

04 11

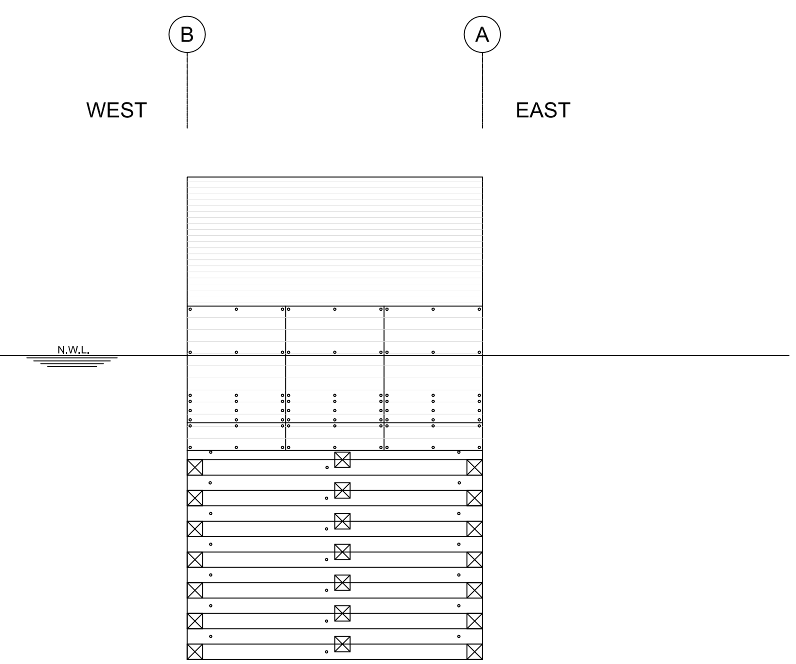
0 10 20 30 40 50 60 70 80 90 100 110 120 130 140 150 160 170 180 190 200 millimetres

LEGEND

LIGHT SCALING	N NARROW CRACK	MEDIUM SCALING	E EFFLORESCENCE
SPALLING	M MEDIUM CRACK	WET AREAS	R RUST STAIN
DELAMINATION	W WIDE CRACK	CONCRETE PATCH	W WET STAIN
HONEYCOMBING	STAINED CRACK	PATTERN CRACKING	N.W.L. NORMAL WATER LEVEL
STAIN	INJ. INJECTION	LIGHT EROSION	U/S DENOTES UNDERSIDE
			ER EXPOSED REBAR
			VOID



CRIB NORTH ELEVATION
1:50



CRIB SOUTH ELEVATION
1:50

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

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

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revision		date
04		
03		
02		
01		

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

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

project title / titre du projet
HAMLET BRIDGE (BRIDGE 57)
COMPREHENSIVE DETAILED INSPECTION
HAMLET ONTARIO

drawing title / titre du dessin
WEST PIER DETERIORATION IV (GUIDE PIER)

drawn by / dessine par: SAS

designed by / conc par: ----

approved by / approuve par: PM

tender / soumission: DUNCAN MANSER — project manager / administrateur de projets

project date / date du projet: MARCH 2012

project no. / no. du projet: 2011-4650-20027340

drawing no. / dessine no.: B-7

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

APPENDIX F

PAINT TESTING RESULTS

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Certificate of Analysis

Delcan(Ottawa)
1223 Michael Street, Suite 100
Ottawa, ON K1J 7T2
Attn: Peter Harvey

Phone: (613) 738-4160
Fax: (613) 739-7105

Client PO:
Project: Hamlet Bridge
Custody:

Report Date: 25-Oct-2011
Order Date: 20-Oct-2011
Order #: 1143205

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1143205-01	Sample #1
1143205-02	Sample #2
1143205-03	Sample #3
1143205-04	Sample #4
1143205-05	Sample #5
1143205-06	Sample #6

Approved By: Mark Foto

Mark Foto, M.Sc. For Dale Robertson, BSc
Laboratory Director

Certificate of Analysis

Report Date: 25-Oct-2011
Order Date:20-Oct-2011

Client: **Delcan(Ottawa)**
Client PO:
Project Description: Hamlet Bridge

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Mercury	EPA 7471A - CVAA, digestion	24-Oct-11	24-Oct-11
Metals	EPA 6020 - Digestion, ICP-MS	24-Oct-11	24-Oct-11

PARACEL
LABORATORIES LTD.

Order #: 1143205

Certificate of Analysis

Client: **Delcan(Ottawa)**

Client PO:

Report Date: 25-Oct-2011

Order Date:20-Oct-2011

Project Description: Hamlet Bridge

	Client ID:	Sample #1	Sample #2	Sample #3	Sample #4
	Sample Date:	29-Sep-11	29-Sep-11	29-Sep-11	29-Sep-11
	Sample ID:	1143205-01	1143205-02	1143205-03	1143205-04
	MDL/Units	Paint	Paint	Paint	Paint
Metals					
Lead	5 ug/g	16700	6710	64900	31800
Mercury	2 ug/g	<2	<2	<2	<2

	Client ID:	Sample #5	Sample #6	-	-
	Sample Date:	29-Sep-11	29-Sep-11	-	-
	Sample ID:	1143205-05	1143205-06	-	-
	MDL/Units	Paint	Paint	-	-
Metals					
Lead	5 ug/g	6060	3690	-	-
Mercury	2 ug/g	<2	<2	-	-

Certificate of Analysis

Client: **Delcan(Ottawa)**

Client PO:

Report Date: 25-Oct-2011

Order Date:20-Oct-2011

Project Description: Hamlet Bridge

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Metals									
Lead	ND	5	ug/g						
Mercury	ND	2	ug/g						

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Metals									
Lead	5290	5	ug/g	3690			35.6	50	
Mercury	ND	2	ug/g	ND				35	

Certificate of Analysis
Client: **Delcan(Ottawa)**
Client PO:

Report Date: 25-Oct-2011
Order Date:20-Oct-2011

Project Description: Hamlet Bridge

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Metals									
Lead	206		ug/L	148	116	70-130			
Mercury	14.0	2	ug/g	ND	93.2	70-130			

Sample and QC Qualifiers Notes

None

Sample Data Revisions

None

Work Order Revisions/Comments:

None

Other Report Notes:

n/a: not applicable
MDL: Method Detection Limit
Source Result: Data used as source for matrix and duplicate samples
%REC: Percent recovery.
RPD: Relative percent difference.

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Page __ of __

Client Name: DELCAN CORPORATION	Project Reference: HAMLET BRIDGE	TAT: [✓] Regular
Contact Name: PETER HARVEY	Quote #	[ ] 2 Day
Address: 1223 MICHAEL STREET OTTAWA, ON, K1N 7X4	PO #	[ ] 1 Day
	Email Address: P.HARVEY@DELCAN.COM.	[ ] Same Day
Telephone: 613-738-4160		Date Required:

Samples Submitted Under: [ ] O. Reg. 153/04 Table ___ [ ] O. Reg 511/09 Table ___ [ ] PWQO [ ] CCME [ ] Sewer Use (Storm) [ ] Sewer Use (Sanitary) [ ] Other: ______

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Paracel Order Number: 1143205

Required Analyses

	Sample ID/Location Name	Matrix	Air Volume	# of Containers	Sample Taken Date	Time	LEAD	MERCURY									
1	SAMPLE #1	P		1	29/9/2011	—	✓	✓									
2	SAMPLE #2	P		1	"	—	✓	✓									
3	SAMPLE #3	P		1	"	—	✓	✓									
4	SAMPLE #4	P		1	"	—	✓	✓									
5	SAMPLE #5	P		1	"	—	✓	✓									
6	SAMPLE #6	P		1	"	—	✓	✓									
7																	
8																	
9																	
10																	

Comments:

Method of Delivery: Walk-in

Relinquished By (Print & Sign): PETER HARVEY Pete Harvey	Received by Driver/Depot:	Received at Lab: S Co	Verified By: S Co
	Date/Time:	Date/Time: Oct 20/11 3:35pm	Date/Time: Oct 20/11 3:45pm
Date/Time:	Temperature: ____ °C	Temperature: ____ °C	pH Verified [ ] By: N/A

APPENDIX G

GEOTECHNICAL INVESTIGATION

REPORT

March 7, 2012

GEOTECHNICAL INVESTIGATION

Hamlet Bridge (Fixed Span) over Trent Severn Waterway Parks Canada Agency Hamlet, Ontario

Submitted to:
Mr. Patrick Mergel P. Eng.
Delcan Corporation
1223 Michael Street, Suite 100
Ottawa, Ontario
K1J 7T2

Report Number: 11-1111-0118

Distribution:

4 copies - Delcan Corporation

2 copies - Golder Associates

Golder Associates

Table of Contents

ATTACHMENTS

APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Delcan Corporation (Delcan) to provide geotechnical foundation engineering services in support of the design of the east abutment of the fixed portion of the Hamlet Bridge located at Canning Road, over the Trent Severn Waterway in Hamlet, Ontario as shown on Figure 1. Authorization to proceed with the investigation was given by Mr. Patrick Mergel, P.Eng. in an email dated September 9, 2011.

This report provides the results of the geotechnical investigation and should be read in conjunction with the *"Important Information and Limitations of This Report"* (attached). The reader's attention is specifically drawn to this information, as it is essential for the proper use and interpretation of this report.

2.0 SITE AND PROJECT DESCRIPTION

The existing bridge structure consists of a swing bridge on the west side and a fixed bridge on the east side. The fixed bridge is steel through truss with a wooden deck. The bridge has an overall length of approximately 303 feet (200 feet for the swing bridge and 103 feet for the fixed bridge). The bridge was built between 1920 and 1922. It is understood that the west abutment (i.e. the abutment for the swing bridge) was reconstructed sometime between 1985 and 1990. The bridge has two (2) intermediate piers. The west pier supports the central pivot to swing the bridge. The east pier supports both the swing bridge and the fixed bridge.

As shown in Pictures 1 and 2 below, the space between the guardrails of the swing bridge and fixed bridge is about 30 mm wider on the north side of the bridge (Picture 1) than on the south side of the bridge (Picture 2). The swing bridge operator has reported that the space between the guardrails of the swing bridge and fixed bridge used to be approximately the same width on both the north and south sides of the bridge, however, the width of the space on the both sides of the bridge have decreased over the past several years. It is inferred from these observations that the foundations of the fixed bridge have moved relative to the swing bridge.

Picture 1: The space between the guardrails of the swing bridge and fixed bridge on the north side; the space was reported to be wider in the past.

Picture 2: The space between guardrails of the swing bridge and fixed bridge on the south side; the space used to be much wider and was approximately the same as the space on the north side. It is now about 30 mm less than the space on the north side.

The west end of the fixed bridge is supported on a set of three steel roller bearings (i.e. sliding bearing) on top of the east pier to accommodate expansion and contraction movements due to temperature changes. The east end of the fixed bridge is fixed to the top of the east abutment. The observed magnitude of movement of the fixed bridge relative to the swing bridge is greater than the expected movement due to expansion and contraction. Further it has been necessary to shorten the wooden bridge deck over the past few years to accommodate this movement.

As shown in the Picture 3 below, the abutment face wall is tilting towards the river.

Picture 3: Note the tilting of the abutment wall and the severe concrete spalling at the lower portion of the abutment wall; a horizontal linear crack through the middle of the entire wall appears to be a construction joint.

As shown in the Picture 4 below, the outwards tilting of the southeast wing wall was also noted.

Picture 4: Note the tilting of the southeast wing wall; the linear crack at the ground surface level appears to be a construction joint.

Based on the information provided by the swing bridge operator (who has been working on the bridge for 20 years), voids/gaps have developed immediately behind the concrete abutment wall in recent years, as shown in the Picture 5 below.

Picture 5: Note the voids/gaps developed behind the abutment wall (i.e. patched area).

The detailed foundation design of the east abutment is not available; however, based on the information provided by the facility maintenance supervisor of Parks Canada, it is understood that the east abutment is supported on spread footings.

Because the east end of the fixed bridge deck is fixed on the east abutment and the west end is supported on the "sliding" bearing on the pier, the reported movement of the fixed bridge deck is likely the result of the movement of the east abutment. The tilting of the east abutment wall may be indicative of the instability of the existing abutment wall and/or abutment footing. However, the tilting may have resulted at the time of the construction as no detailed construction records are available for review.

No drainage holes were observed in the east abutment wall and the wing walls.

3.0 INVESTIGATION PROCEDURE

The field work for this investigation was carried out on September 28 and October 22, 2011, during which time three boreholes, two coreholes and one test pit were advanced at the locations shown on the Borehole and Test Pit Location Plan, Figure 2 and Photos 3 and 4 on Figure 13B. Two boreholes (Borehole 1 and 2) located behind the east abutment wall were drilled using a truck-mounted drillrig supplied and operated by a drilling specialist, under our supervision. Standard penetration testing and sampling were carried out at regular intervals of depth in the boreholes using conventional 35 mm internal diameter split spoon sampling equipment. Rock coring using NQ-sized rock coring equipment was carried out in Borehole 2. In addition, in situ vane shear tests were carried out in the relatively soft clayey soils encountered in the boreholes. Based on the information provided by a private locater, there is a traffic signal underground cable located immediately along the north wing wall, and therefore, no test pit was carried at the north wing wall location. Test Pit 1 was hand excavated by Golder staff at a location immediately south of the southeast wing wall. A horizontal corehole was advanced on the west face of the east abutment wall using a coring machine supplied and operated by a coring specialist, working under our supervision. A vertical corehole was advanced from the top of the concrete abutment wall through the entire height of the abutment wall. Borehole 3 was carried out through the vertical corehole and two soil samples were recovered immediately below the base of the existing concrete abutment.

The shallow groundwater conditions were noted in the open boreholes/test pit during drilling/test pitting. A standpipe piezometer was installed in Borehole 2 to allow for further monitoring of the shallow groundwater levels.

All of the soil samples, concrete cores and rock core samples obtained during this investigation were brought to our Whitby and Mississauga laboratories for further examination, natural water content testing, organic content testing, selected classification testing, unit weight testing and compressive strength testing.

The field work for this investigation was directed by members of our engineering staff who also determined the borehole/corehole/test pit locations in the field, logged the boreholes/coreholes/test pit, and cared for the samples obtained.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The bridge site is located in the area between physiographic regions known as the Georgian Bay Fringe and The Number 11 Strip, according to *The Physiography of Southern Ontario* (Chapman and Putnam, 1984). The general area is located at the southern fringe of the Canadian Shield and underlain by Precambrian rocks which can be classified as biotite, gneiss, schist and granite. The glaciated rock surface on land is quite undulating and features numerous ridges, knobs and erratic outcrops, which is consistent with the results of the investigation.

4.2 Subsurface Conditions

The existing subgrade soils and shallow groundwater conditions encountered in the boreholes and test pits, as well as the results of the field and laboratory testing, are shown in detail on the Record of Borehole and Record of Test Pit sheets, following the text of this report. Lists of abbreviations and symbols are provided to assist in the interpretation of the borehole logs.

It should be noted that the boundaries between the strata shown on the borehole/test pit logs have been inferred from drilling/coring/test pitting observations and non-continuous samples. They generally represent a transition from one soil type to another and should not be inferred to represent an exact plane of geological change. Further, conditions will vary between and beyond the boreholes/test pit. The following is a summarized account of the subsurface conditions encountered in the boreholes drilled at the site, followed by more detailed descriptions of the existing fill and native soil strata, and shallow groundwater conditions.

The subsurface soil conditions generally consisted of shallow fill overlying very soft to soft silty clay underlain by very loose to dense sandy silt to silty sand underlain by bedrock at a depth of about 8.0 m below ground surface.

Pavement Structures

Pavement structure was encountered surficially in Boreholes 1 and 2. The pavement structure consisted of 80 mm and 85 mm of asphalt overlying about 270 mm to 310 mm of granular base.

Abutment Wall Concrete

Concrete was encountered from ground surface to about 3.5 m (i.e. the abutment wall height at this location) below ground surface in the vertical corehole as shown in Photos 10 to 12 on Figures 13E to 13F. Two concrete core samples obtained from the vertical corehole were tested for compressive strength according to CSA A23.2 C14 as shown on Figure 9. The measured compressive strengths were 18.2 MPa and 14.2 MPa.

Concrete was encountered in the horizontal corehole carried out on the west face of the abutment wall as shown in the Photos 7, 13 to 15, on Figures 13D, 13G and 13H. The horizontal corehole was located on the existing linear crack (i.e. construction joint). As noted in the photos, the linear construction joint is generally horizontal and runs through the entire corehole length. Based on the water stains observed from the construction joint surfaces, the concrete appears to be separating along most of the construction joint likely due to erosion along the joint. Some vertical cracks were observed in the retrieved cores. The length of the corehole is approximately 1.7 m (i.e. inclined backwall). The concrete wall is about 1.2 m wide at the ground surface.

Topsoil

Topsoil was encountered surficially in Test Pit 1. The thickness of the topsoil was 120 mm.

Fill Materials

Fill materials were encountered in Boreholes 1 and 2 and in the test pit. The fill extended to depths of 2.1 m and 2.4 m below ground surface in Boreholes 1 and 2. The fill extended to a depth of 0.9 m below ground surface in Test Pit 1. Test Pit 1 was terminated in the fill materials when large diameter rock fill was encountered.

The fill materials encountered in the boreholes are associated with previous backfilling behind the east abutment wall and wing walls. The fills are variable in composition but generally consist of silty sand, fine sandy silt and organic silt. The fill materials on the south side of the south wing wall are associated with previous backfilling and generally consist of variably sized rock fragments with silty sand.

Standard penetration tests carried out within the fill materials gave "N" values ranging from 3 blows to 11 blows per 0.3 m penetration, indicating a very loose to compact relative density. The in-situ water contents of the fill samples ranged widely from 2 percent to 44 percent.

Grain size distribution curves for the fill samples of the silt sand, sandy silt and organic silt fills are shown on Figures 3, 4 and 5. Based on the grain size distribution, the sandy silt fill and organic silt fill are considered to be of highly frost susceptibility due to the high silt content of these materials.

Clayey Silt to Silty Clay

Clayey silt to silty clay was encountered in all boreholes below the fill materials or concrete abutment and extended to depths ranging from 4.0 m to 5.3 m below the existing ground surface. Borehole 3 was terminated in the silty clay. Standard penetration tests carried out within the clayey silt to silty clay gave "N" values of 2 blows and 5 blows per 0.3 m, indicating a very soft to firm consistency. In-situ vane testing carried out within the clayey silt to silty clay gave undrained shear strength and remoulded undrained shear strengths ranging of 20 kPa to 38 kPa and 2 kPa to 10 kPa, respectively, indicating a soft to firm consistency and low sensitivity. The natural water contents of the clayey silt to silty clay samples ranged from 23 percent to 31 percent.

Unconfined compression testing (ASTM D 2166-06) was carried out on a clayey silt sample recovered from the core barrel at Borehole 3. The measured unit weight of the clayey silt sample was 19.6 kN/m^3. The undrained shear strength inferred from the compression test is 47 kPa, indicating a firm consistency.

The results of grain size distribution of samples of clayey silt are provided on Figure 6. Based on the grain size distribution, the clayey silt samples are considered to be of high frost susceptibility.

The results of Atterberg limit tests performed on three samples of clayey silt are provided on Figure 8 and also summarized on the Record of Borehole sheet. The result of these laboratory tests on the samples of clayey silt indicated the liquid limits of 30, 35 and 41 percent, plastic limits of 21, 21 and 20 percent, and plasticity index ranging from 9, 14 and 21, respectively.

Organic Silt

A deposit of organic silt was encountered in Borehole 1 below the clayey silt to silty clay and extended to a depth of 5.6 m below the existing ground surface. One standard penetration test carried out within the organic silt gave an "N" value of 0 blow (i.e. weight of hammer) per 0.3 m, indicating a very loose relative density. The natural water content of the organic silt sample was 32 percent. The organic contest measured from the organic silt sample was 2.6 percent as shown on Figure 12.

Fine Sandy Silt to Silty Fine Sand

Fine sandy silt to silty fine sand was encountered in Borehole 1 and 2 below the silty clay/clayey silt or organic silt and extended to depths of 7.1 m and 6.4 m below the existing ground surface. Standard penetration tests carried out within the fine sandy silt to silty fine sand gave "N" values of 1 blow and 4 blows per 0.3 m, indicating a very loose to loose relative density. The natural water contents of the fine sandy silt to silty fine sand samples ranged from 23 percent to 32 percent.

Sand/Silty Sand and Gravel

Sand and silty sand and gravel were encountered in Borehole 1 and 2 below the silty fine sand and extended to depths of about 7.9 m below the existing ground surface. The sand/silty sand and gravel deposits contain cobbles and boulders. Standard penetration tests carried out within the sand/silty sand and gravel gave "N" values of 13 blows and greater than 100 blows per 0.3 m, indicating a compact to very dense relative density. The natural water contents of the sand/silty sand and gravel samples were 10 percent and 12 percent. The result of grain size distribution of a sample of sand is provided on Figure 7.

4.3 Bedrock

Bedrock was encountered in Borehole 1 and 2 below the sand/silty sand and gravel. This bedrock was confirmed by coring in Borehole 2 to a depth of 9.4 m below the existing ground surface. Based on the results of rock coring, the bedrock at the site generally consists of fresh to slightly weathered, dark grey to black, fine to medium grained, biotite gneiss bedrock.

The Total Core Recovery (TCR) of the core samples in Borehole 2 was 98 percent; the Solid Core Recovery (SCR) was 97 percent; and the Rock Quality Designation (RQD) was 93 percent. Based on these results and on our visual examination of the core samples, the rock quality of the biotite gneiss encountered is generally considered to be excellent.

One sample of the rock core from Borehole 2 was prepared and subjected to compressive strength testing. This testing was carried out in general accordance with ASTM Standard Test Method D 7012-07, entitled "Standard Test Method for Unconfined Compressive Strength of Intact Rock Core Specimens". This testing gave an unconfined compressive strength value of 136.4 MPa, indicating that the strength of this bedrock is classified as very strong (Canadian Foundation Engineering Manual, 2006, 4th Edition, Table 3.5).

4.4 Shallow Groundwater

Shallow groundwater was encountered in Boreholes 1 and 2 upon completion of drilling at depths of about 4.1 m below the existing ground surface. Test Pit 1 was dry upon completion. The groundwater level subsequently measured in the piezometer installed in Borehole 2 was at a depth of 1.9 m below the existing ground surface, on October 22, 2011.Details of our groundwater level observations are shown on the Record of Borehole Logs and Test Pit sheets, which follow the text of this report.

The groundwater levels at the site should be expected to fluctuate seasonally in response to changes in precipitation and snow melt, and should be expected to be higher during the spring season or during any period of heavy precipitation. The groundwater levels at the abutment area should be influenced by the water level in the river. The water level in the river was about 2.9 m below the bridge deck when measured on September 28, 2011.

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation recommendations for the proposed remediation of the existing bridge foundations. These recommendations are based on our interpretation of the factual data obtained from the boreholes and the test pit advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives in support of the remedial design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Foundation Options

Based on the information provided by the facility maintenance supervisor from Parks Canada, it is understood that the existing east abutment is supported on shallow foundations. Based on the borehole/corehole data, the native soil deposits at the underside level of the existing abutment wall foundation (i.e. 3.5 m below ground surface) generally consist of very soft to soft clayey silt to silty clay. It is noted that the measured undrained shear strength of the soil directly below the existing abutment foundation is higher than the measured undrained shear strengths of the soils beyond the abutment wall footprint. The relatively higher undrained shear strength of the soil immediately underneath of the abutment wall foundation is a result of the consolidation process that has occurred in this soil due to the loads imposed by the abutment foundation. The total loading of the existing abutment is not available, however, based on the existing soil conditions, the observed tilting of the abutment wall has likely resulted from overstressing of the founding soils and it is unlikely that the cause of the observed tilting is poor construction technique.

Possible remedial works would likely consist of the full replacement or underpinning of the existing east abutment. A summary of the advantages and disadvantages associated with the various foundation options are provided below.

- **Strip or spread footings founded on native deposits:**

  Based on the observations noted above, the existing east abutment is currently moving. The native loose/soft soils at the founding level of the existing abutment are not suitable to support new shallow foundations for a replacement abutment. Underpinning of the existing abutment would need to extend below these soils to encounter competent founding material at depth, as noted below.

- **Steel Piles driven to found on the underlying shallow bedrock for abutment replacement:**

  Steel pipe piles or H piles driven to refusal on the bedrock may be used for support of the new abutment. Due to the relatively high strength of the bedrock at surface, the difficulties in socketing the piles into the bedrock should be anticipated, especially for battered piles. This option will require the mobilization of large piling equipment. The soft/loose soils at the abutment location may not support the heavy equipment loads and significant site preparation may be required to construct a working platform for the pile driving equipment. The vibrations caused by the pile driving would need to be monitored to ensure damage is not caused to the bridge structure and nearby buildings.

- **Micropiles driven to found into the underlying shallow bedrock for both underpinning and new construction:**

  Considering the subsurface conditions identified during the field investigation, micropiles cased through the overburden and having the (uncased) bond zone socketed into the good to excellent quality bedrock at depth, is considered to be a feasible solution for this site. The micropile installation would not create significant vibrations during installation. The installation equipment is relatively small and is better suited to the limited working area at the site. The micropiles could be considered for both underpinning of the existing abutment and for a full abutment replacement.

- **Helical Piles founding on the underlying shallow bedrock for underpinning:**

  Considering the subsurface conditions, helical piles screwed through the overburden and founded on top of the bedrock, are considered to be a feasible solution for underpinning the existing abutment. The helical pile installation would not cause significant vibrations during installation. The installation equipment is relatively small and is better suited to the limited working area at the site. The helical pile is considered to be more appropriate for underpinning the existing abutment than for supporting a replacement abutment.

5.3 Abutment Replacement Supported on Driven Piles

5.3.1 Driven Piles

End bearing driven piles may be considered for the new foundations, if the full replacement of the east abutment and wing wall is the preferred remedial option. This foundation option would eliminate the need to excavate and remove the very soft/very loose soil deposits beneath the abutment and wing wall footprints. Based on the borehole data, the end bearing piled foundations should be driven to refusal on the underlying bedrock surface, at depths of about 8.0 m below the existing ground surface.

Boulders and cobbles should be expected to be present above the bedrock. Based on the identified soil conditions, a thickly walled, high stress steel, open end pipe pile foundation may be considered for the proposed abutment replacement. Pipe piles are available in a variety of sizes with various wall thicknesses. It is recommended that the minimum outside diameter pipe considered for this project should be 219 mm (8 5/8 inches), as smaller diameter pipes tend to bend significantly during driving. The piles should have a minimum wall thickness of 12 mm (1/2 inch) or greater to minimize the bottom damage as they are driven into the bedrock. Additional thickness may be considered for protection against potential corrosion. Open end pipe piles have the advantage that they are easier to key into sloping bedrock, if encountered.

The factored bearing resistances at Ultimate Limit State (ULS) for open end pipe piles in the conditions at this site should be limited to 0.3 times of the yield strength of the steel multiplied by the available cross sectional area of the pipe wall (i.e. the area of the steel cross section not the area of the pipe pile cross section). This will reduce the required driving stresses, and therefore, reduce the potential pile damage due to high localized bottom stresses as a result of driving to key into the bedrock. Piles driven to refusal on bedrock can be designed based on the following:

> Factored Axial Bearing Resistances at ULS = Cross Sectional Area of Pipe Wall (A_s) x 0.3 x Yield Stress (i.e. F_v=350 MPa)

Steel H piles may also be considered, although from our experience, the open end high stress steel pipe piles are more favourable because they more easily "bite" into the bedrock surface and are better for penetrating through cobbles and boulders. However, should the H piles be selected, the above axial bearing resistance calculation should be reduced to 75% to account for H piles bearing on bedrock. The factored axial bearing resistance at ULS for various H pile dimensions are as follows:

H Pile Dimensions	Factored Axial Bearing Resistance at ULS
HP310x79	800 kN
HP310x10	1100 kN
HP310x132	1200 kN

This calculation assumes that the piles are driven to practical refusal with a minimum pile set of 10 blows per 25 mm (or as otherwise determined by the electronic Pile Driving Analyzer). The settlement of the individual pile and the pile group at this pile load is anticipated to be less than 25 mm. The geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistances at ULS, and therefore, the factored axial resistances at ULS should govern the structural design.

It is recommended that piles should have a minimum spacing of at least 3.5 pile diameters, centre to centre. Although open end piles are low displacement piles, it is still possible that driving of piles in a large group will result in heaving of previously driven piles. If this pile heaves exceed 5 mm (¼ inch), the previously driven piles should be re-struck to at least the required driving resistance.

Because of potential boulder obstructions at depth, “seating” of the piles within the normally accepted tolerances in the piling industry (2% plumbness and 75 mm location) might be difficult to achieve. In this case, it is suggested that the potential of some piles exceeding these tolerances up to at least 4% plumbness and 150 mm (6 inches) for location may be considered in the structural design.

Conventional pile driving operations may cause vibrations that could cause some movement of soils in the surrounding area, which could potentially induce settlements of any nearby shallow foundations beneath adjacent structures (i.e., if they are not on piled foundations or shallow foundation on bedrock). Careful monitoring records of the piling operation and monitoring of the surrounding structures during piling are recommended. An evaluation of existing surrounding foundation types and a pre-construction condition survey should be carried out prior to pile driving operations.

According to the Ontario Building Code, the pile installation should be monitored full-time by the design engineers or their representatives. This is especially important with the founding conditions at this site, as the piles must be driven to develop their axial bearing resistances on bedrock, where bottom damage may occur. Consequently, Golder should be retained to monitor pile installation, as well as to review the design drawings and specifications prior to tendering. Otherwise, Golder cannot be responsible for the performance of the piles.

For steel pipe piles, following successful installation of the piles, it is recommended that the interior of each pipe pile be pumped out and the open upper section filled with concrete. Although the concrete will not provide any additional bearing capacity, it will increase the resistance to bending and reduce potential corrosion from groundwater acting on the inside wall of the piles.

The driving energy and final "set" criteria is normally developed by the piling contractor. However, from our previous experience, we would normally suggest that the end bearing piles be driven with a hammer developing energy of not less than 50 kJ (39,000 foot pounds). A minimum final "set" criteria of ten blows per 25 mm (1 inch) penetration for a minimum of three consecutive series of 10 blows should be adopted for the smaller HP310x79 and HP310x110 piles, with the Pile Driving Analyser used to verify the pile capacity and required set criteria. A more stringent set criteria of 20 blows/25 mm (25 blows/inch) may be necessary for the larger HP310x132 piles and 219 mm and 245 mm pipe piles, unless proven otherwise in the field. The "set criteria" on each pile must be confirmed by a qualified, full-time, piling inspector.

If some of the piles meet refusal at a relatively shallow depth compared to adjacent piles, consideration should be given to either removing the pile and driving a new pile approximately 0.5 to 1.0 m away, or verify the capacity of the pile with the Pile Driving Analyser. Deviations from the design pile layout must be discussed with and approved by the structural design engineer.

To minimize seating difficulties and to protect the pile tips from damage, it is recommended that the piles be provided with driving shoes (reinforced tips) to minimize damage to the pile during driving and penetration through the cobbles and boulders overlying the bedrock.

Subsoil movement during pile driving should also be considered, especially for the pile driving adjacent to the existing sheet pile walls. Close control over final pile tip elevations must be maintained and re-tapping of selected piles may be necessary. Also, during winter pile driving operations, energy losses of 20% or more can be anticipated. These losses should be considered during selection of the pile driving equipment and during driving operations.

Settlement of any structure, founded on the end bearing piles driven to practical refusal in the underlying bedrock, should be negligible and would basically consist of the elastic compression of the pile member. It is recommended that the final grade should not be increased and the average unit weight of the backfill materials adjacent to piles should not be more than the average unit weight of existing fills. Otherwise, additional loading may occur, that would cause additional stresses in the very soft/very loose soil deposits surrounding the piles. The additional stresses would cause settlement as well as negative skin friction on the piles, which should be considered in the structural designs.

5.4 Micropile Option for Abutment Replacement and Underpinning

As an alternative to the conventional pile option as discussed above, consideration may be given to a micropile option for the proposed abutment replacement or underpinning. Considering the loose/soft nature of the overburden deposits, it would not be practical to attempt to design the micropiles to be bonded only within the overburden to support the abutment replacement or underpin the existing abutment.

Design loads have not been provided, however, considering the potential for the loads to include combinations of axial and lateral load as well as bending moments and considering the availability of typical casing and bar sizes, the following two micropile sections are provided as options for the support of the abutment replacement or underpinning:

Option	Outer Steel Casing		Inner Steel Reinforcement	
	(metric)	*(imperial)*	*(metric)*	*(imperial)*
#1	HSS 273 x 13	10-3/4" x 1/2" wall	57 (bar)	#18 (bar)
#2	HSS 194 x 12	7-5/8" x 1/2" wall	43 (bar)	#14 (bar)

It should be noted that the effects of the smaller cross-section on the lateral stiffness of the micropile must be considered from a structural point of view.

5.4.1 Micropile Design Assumptions

The micropile design is based on the premise that the foundation loads will have to be fully supported by the micropiles (no contribution from the abutment footing) due to concerns over the soft/lose nature of the founding soils and potential minor ongoing consolidation settlement of the soft clayey deposits.

The design is based on the approach outlined in the FHWA/NHI Micropile Design and Construction Reference Manual, Publication No. FHWA NHI-05-039 (FHWA/NHI 2005).

In all cases, the centre-to-centre spaces between individual micropiles should be at least 820 mm (32 inches) or three micropile diameters, whichever is greater. For micropiles with the bond zone in competent bedrock, it is assumed that a group reduction factor is not required. Further, if the bond resistances between the micropile and the surrounding soils are considered in the structural design, the efficiency factor (η) for the micropile group should be in accordance to the Table 5-4 (FHWA/NHI 2005) for 'cap not in firm contact with the ground and the ground is relatively soft (undrained shear strength less than 95 kPa)'.

A Factor of Safety of 2.0 for the geotechnical capacity of the micropile is to be used based on the recommendations in Section 5.9.2 (FHWA/NHI 2005), considering the micropile bond zone will be formed in the competent very strong bedrock and assuming that at least one verification test will be conducted prior to commencing production micropile installation.

In consideration of the potential for aggressive ground conditions at the site, it is recommended to have a minimum 1.6 mm section loss (all around) be included in the design of the outer casing of the micropiles.

5.4.2 Casing and Central Bar

As noted above, a micropile comprised of an outer HSS 273 x 13 (10-3/4" x 1/2" wall) casing with an inner 57 mm or #18 (2-1/4") bar and a micropile comprised of an outer HSS 194 x 12 (7-5/8" x 1/2" wall) casing with an inner 43 mm or # 14 (1-3/4") bar have been considered as options for supporting the abutment at this site.

The following sections describe the details of the micropile design.

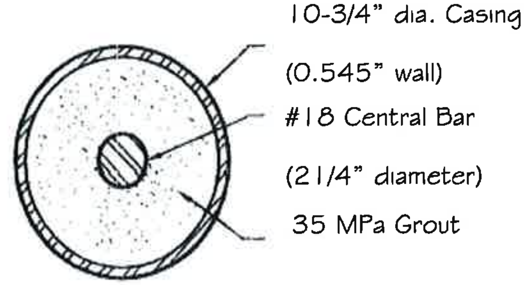
5.4.3 Micropile Design Details

Following the procedures outlined in the Micropile Design and Construction Reference Manual (FHWA/NHI 2005), it is recommended that the micropiles be comprised of the following components, steel grades and grout strength.

Option 1:

Steel Casing:

- API-N80 (threaded)
- 80 ksi, F_y = 552 MPa
- 10-3/4" (273 mm) outside diameter
- 0.545" (13.84 mm) wall thickness



Central Bar:

- Dywidag GEWI Threadbar (or equivalent)
- 75 ksi (Grade 500), Fy = 520 MPa
- #18 bar
- 2.25" (57 mm) diameter

Grout:

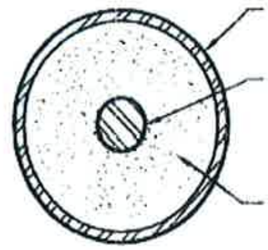
- 35 MPa (minimum at 28 days)

Water/Cement Ratio (by weight) < 0.45

Option 2:

Steel Casing:

- API-N80 (threaded)
- 80 ksi, F_y = 552 MPa
- 7-5/8" (194 mm) outside diameter
- 0.545" (13.84 mm) wall thickness



Central Bar:

- Dywidag GEWI Threadbar (or equivalent)
- 75 ksi (Grade 500), Fy = 520 MPa
- #14 bar
- 1.75" (43 mm) diameter

Grout:

- 35 MPa (minimum at 28 days)
- Water/Cement Ratio (by weight) < 0.45

In order to develop the axial geotechnical resistance, the micropiles will have to be socketed into the good to excellent quality bedrock. For design purposes, it is recommended that the outer steel casing extend at least 1.5 m below the top of bedrock (i.e. casing plunge length = 1.5 m). This plunge length will need to be confirmed by a lateral pile group analysis performed by the structural engineer based on the recommendations for the resistance to lateral loading provided in the later section of this report. At least one verification pile loading test should be carried out prior to commencing production micropile installation.

5.4.4 Axial Geotechnical Resistances

The axial geotechnical resistance of the micropiles will be primarily developed within the bond zone or the uncased lower section of the micropile socketed into the bedrock.

The grout-to-ground bond strength in the bedrock has been estimated based on the results of the tests performed on specimens of the bedrock core and from recommended values for bedrock found in Micropile Design and Construction Reference Manual (FHWA/NHI 2005). Based on this information, a grout-to-ground/bedrock ultimate bond value (α_{bond}) of 1,500 kPa could be used for design. This value will have to be verified by the pre-production micropile load tests to be conducted at the site prior to the start of production piling.

Although the cased section of the pile will have a nominal diameter of 0.273 m (10-3/4") or 0.194 m (7-5/8"), the uncased section of the pile in the bond zone within the bedrock will likely have a smaller diameter as a result of the method of installation and drilling that is likely to be adopted by the contractor. It is likely that after advancing the casing to the required depth within the bedrock (i.e. at least 1.5 m as noted above), the contractor will seat the casing and then drill the bond zone below this depth, creating an open hole in the rock with a smaller diameter than the cased hole. For the purposes of design, it is assumed that the bond zone will have a minimum diameter of 0.229 m (9") and 0.15 m (6").

For a micropile comprised of an HSS 273 x 13 (10-3/4" x 1/2" wall) casing, with an inner 57 mm or #18 (2-1/4") bar and a minimum diameter of 0.229 m (9") in the bond zone socketed into the good to excellent quality bedrock, for a 1.5 m grout-to-ground/bedrock bond zone (i.e. below the plunge zone/casing), the factored axial geotechnical resistance at ULS of 800 kN may be considered in the design. The resistances of the plunge length (i.e. cased length below the bedrock surface) should be ignored in the design. The resistances of the cased length above bedrock should also be ignored during the design, because the frictional resistances may not be able to be mobilized based on the anticipated elastic compression of the micropiles.

For a micropile comprised of an HSS 194 x 12 (7-5/8"x 1/2" wall) casing, with an inner 43 mm or #14 (1-3/4") bar and a minimum diameter of 0.15 m (6") in the bond zone socketed into the good to excellent quality bedrock, for a 1.5 m grout-to-bedrock bond zone (i.e. below the plunge zone/casing), the factored axial geotechnical resistance at ULS of 500 kN may be considered in the design. The resistances of the plunge length (i.e. cased length below the bedrock surface) should be ignored during the design. The resistances of the cased length above bedrock should also be ignored during the design, because the frictional resistances may not be able to be mobilized based on the anticipated elastic compression of the micropiles.

The geotechnical resistance at SLS for 25 mm of settlement (for the length of piles required at this site) will be greater than the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, factored ULS conditions will govern the design for this foundation type.

5.4.5 Design and Installation Considerations

Buckling

Given the soft/loose nature of the soil deposits below the proposed abutment foundation, the potential for buckling of the proposed micropile cross-section within the soft/loose soil deposits should be checked by the structural engineer considering the combination of axial and lateral loads and bending moments that will be acting on the group.

Combined Axial Compression and Bending of Cased Length

The axial loads and the additional compressive stresses due to lateral loads and/or bending moments imposed on the pile group should be considered in the evaluation of the structural capacity of the cased and uncased sections of the micropiles by the structural engineer.

Corrosion Protection

A steel section loss of a minimum of 1.6 mm of the wall thickness should be considered when evaluating the structural capacity of the micropiles. This section loss is the minimum recommended for sacrificial corrosion protection in the design of the casing (DFI 2004) considering the potentially aggressive ground conditions.

Corrosion protection of the central bar will be provided by specifying epoxy coating combined with grout cover.

Drilling Requirements

The contractor must select a drilling method that will minimize the potential for ground loss and disturbance to the existing foundations during the advancement of the micropiles through the very soft clayey deposits and loose sandy/silty deposits (especially the through cobbles and boulders) to minimize the risk of surface settlement and further movement of the existing abutment/wing wall foundations and existing sheet piles/scour protection. In this regard, it is important that duplex drilling techniques, with the cuttings returning up the inside of the casing, be utilized to advance the holes.

Micropile Connection at Pile Cap

The connection between the top of the micropiles and the reinforced concrete pile cap should be designed to transfer both tension and compression loads. In addition, it is recommended that both the outer casing and inner central bar be extended into the pile cap to accommodate the load and moment transfer. The actual required dimensions and thicknesses of the embedment, bearing plate and stiffener plates will have to be calculated by the structural engineer.

Based on the concrete coring results, the concrete exhibited relatively poor quality, including uneven distribution of aggregate sizes and lack of aggregates in some sections of the cores. Some vertical cracks were also observed in the horizontal concrete cores. The compressive strengths of the concrete cores from the existing abutment ranged from about 14 MPa to 18 MPa, which should be considered in the structural design for underpinning as it may affect the underpinning strategy.

Micropile Grouting

Because the grout is such a vital component of the micropile, close attention must be paid to the control and quality of the product. A grout quality control plan, at a minimum including cube or cylinder compression testing and grout density (water/cement ratio) testing, should be included and reviewed by the engineer prior to the construction.

Type A micropile grout placement techniques (i.e. gravity fill placement techniques) by tremie methods are considered to be feasible for the cased micropile installation as discussed above. The grouting process should be inspected by the geotechnical engineer to ensure the quality.

5.5 Downdrag Load (Negative Skin Friction)

If any grade raise or widening of the embankment is to be placed around the perimeter of the abutment, the additional loading to the very soft clayey silt will cause downdrag loads on the piles. The estimated unfactored downdrag load acting on the piles at this location may be taken as 85 kN per pile for preliminary design, but this additional load should be checked at detail design. The structural capacity of the piles must be checked for the factored dead loads and downdrag loads in accordance with Section C6.8.4 of the CHBDC Commentary for ULS conditions.

5.6 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered piles. In the case of battered piles, precautions during driving are necessary in some situations (such as for specific soil/bedrock conditions/pile lengths and where the batter is shallower than 6 vertical to 1 horizontal) to ensure that the piles do not deflect along the bedrock surface even with relatively flat-lying bedrock. It is recommended that the pile batter be restricted to 1H:8V or steeper for driving piles.

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance.

For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case. According to Broms (1964) potential for 'short' pile failure and 'long' pile failure should be reviewed by the structural designer. For potential 'short' pile failure, the lateral capacity of the soil adjacent to the pile is fully mobilized; for the 'long' pile failure, the bending resistance of the pile is fully mobilized (Canadian Foundation Engineering Manual (CFEM 2006)).

At Serviceability Limit States (SLS), the horizontal resistance of the piles will be controlled by deflections of the pile heads which may be too large to be compatible with the superstructure. In this case, the horizontal resistance in front of the vertical pile may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction (k_h) is determined based on the equations given below (CFEM 1992[1] as noted in CHBDC C6.8.7.1):

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where
k_h is the coefficient of horizontal subgrade reaction (MPa/m);
n_h is the constant of subgrade reaction (MPa/m);
z is the depth (m); and
B is the pile diameter / width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where
k_h is the coefficient of horizontal subgrade reaction (MPa/m);
s_u is the undrained shear strength of the soil (MPa); and
B is the pile diameter / width (m).

The following ranges for the values of n_h and s_u may be assumed in the structural analyses. The soil stratigraphy has been generalized and the range in values reflects the variability in the subsurface conditions within the abutment footprint.

Structure	Soil Unit	n_h (MPa/m)	s_u (kPa)
East Abutment	Very soft Silty Clay below Groundwater Table From Depth 3.4 m to 5.3 m	-	20
	Very loose to compact Sandy Silt to Silty Sand below Groundwater Table From Depth 5.3 m to 8.0 m	10	-

The upper zone of soil (down to a depth below the pile cap equal to about 1.5 x D after Broms 1964, where D = pile diameter) should be neglected in the calculation of lateral resistance of the pile to account for disturbance effects during driving.

Group action for lateral loading from soil resistances should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient

[1] Canadian Foundation Engineering Manual, 1992, 3rd Edition

of horizontal subgrade reaction in the direction of loading by a reduction factor, R (NAVFAC DM-7.2, 1982) as follows:

Pile Spacing in direction of Loading (D = Pile Diameter)	Subgrade Reaction Reduction Factor (R)
8D	1.0
6D	0.7
4D	0.4
3D	0.25

For bedrock:

For the given loads, the rock is expected to remain in the elastic range; therefore, closed form solutions have been used for the estimation of the ground lateral spring constant.

Based on the assessed lateral rock mass elastic modulus of the gneiss bedrock, E_h = 34,000 MPa, and a Poisson's ratio of 0. 2, the lateral rock mass spring constant is given by:

$$k_h = \frac{4\pi(1-\nu)E_h}{(3-4\nu)(1+\nu)}\frac{1}{\ln(r_o/r_i)} = \frac{4\pi(0.8)(34{,}000)}{(2.2)(1.2)}\frac{1}{\ln(10)} = 56{,}200 \quad \text{MN/m/m}$$

Where:

r_i = radius of micropile

r_o = radius of 'zero' deformation; typically 10 to 15 pile diameters.

The passive resistance for the micropile bedrock sockets were analysed using the RMR assessment of the bedrock which utilizes the bedrock strength measurement obtained from the UCS testing on the recovered rock core. The ultimate lateral capacity of the bedrock is estimated to be 67 MPa. The capacity per metre length of micropile within the bedrock in kN can be determined by multiplying the Ultimate Lateral Capacity given above by the diameter of the micropile. A factor resistance of 0.5 should be applied to the ultimate capacity for design.

5.7 Helical Pipe Options

As an alternative to micropiles, helical pipes founded on bedrock may be considered for the underpinning of the existing abutment foundation. The friction from the fill and loose/soft native material should be ignored in the design of the helical piles. An adequate factor of safety should be considered in the design. The steel piles must be provided with adequate corrosion protection. A load test should be carried out by the specialist contractor/supplier to verify the load capacity of the helical pile system in accordance with the requirements of ASTM D-1143, "Method of Testing Piles under Static Axial Compressive Load". The minimum test load shall be twice the superimposed working load. A monitoring program should be in place to confirm that the design pile capacity is achieved.

The actual design details of the helical piles are typically provided by the piling contractor. The specialist contractor's system, installation procedures, proposed load test and monitoring program should be submitted to the Engineer for review and approval. We note that some difficulty may be encountered in advancing the piles through the native soils on top of the bedrock due to the potential presence of cobbles and boulders. However, should an obstruction be encountered, the pile may be extracted and reinstalled at an alternate location.

5.8 Seismic Site Coefficient

Based on the results of site investigation, subsoil conditions encountered behind the existing east abutment generally consists of granular fill overlying clayey silt to silty clay overlying silt/sandy silt to silty sand overlying the gneiss bedrock. For seismic design purposes, the Site Coefficient, S, for the existing east abutment supported on shallow foundation may be taken as 1.5 consistent with Soil Profile Type III in accordance with Section 4.4.6 of the CHBDC (2006). If the existing east abutment is underpinned or replaced to be supported on deep foundations as discussed in Sections 5.3 and 5.4, the Site Coefficient may be taken as 1 consistent with Soil Profile Type I for the east abutment.

According to Section 4.4.4 of the *Commentary to the CHBDC (2006)*, this site is located in Seismic Performance Zone 1 and the zonal acceleration ratio (A) is 0.05 for the Hamlet area. Based on experiences for the subsurface conditions at this site, a 30 per cent amplification of the ground motion may occur during earthquake, resulting in an increase in the ground surface acceleration from 0.05g to be 0.065g. Therefore, the site-specific zonal acceleration ratio (A) for the site is 0.065 and the seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.065.

5.9 Lateral Earth Pressures for Design of Abutment and Wing Walls

The lateral earth pressures acting on the abutment stems and any associated wing walls / toe walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment stems and walls. It should be noted that these design recommendations and parameters assume level backfill at ground surface behind the walls (Case I and II). Where there is sloping ground behind the walls at any other inclination the coefficient of lateral earth pressure must be adjusted to account for the slope. The existing east abutment can be evaluated based on the conditions of Case I described below.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) 1010 Granular ‘A’ or Granular ‘B’ but with less than 5 per cent passing the 200 sieve should be used as backfill behind the abutments and walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the abutment/wall stem, in accordance with *CHBDC (2006)* Section 6.9.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.2 m behind the back of the abutment/wall stem (Figure C6.20(a) of the *Commentary to the CHBDC (2006)*) for the Case I condition or within the wedge-shaped zone defined by a line drawn at 1.2 horizontal to 1 vertical (1.2H:1V) extending up

and back from the rear face of the footing (Figure C6.20(b) of the *Commentary to the CHBDC (2006)) for* the Case II condition.

- For Case I, the pressures are based on the existing abutment fill materials placed with flat ground surface behind the wall and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.36
At rest, K_o	0.53
Passive, K_p	2.77

- For Case II, the pressures are based on the granular fill as placed with flat ground surface behind the wall and the following parameters (unfactored) may be assumed:

	Granular "A"	**Granular "B" Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, the at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as follows:
  - Rotation of approximately 0.002 about the base of a vertical wall;
  - Horizontal translation of 0.001 times the height of the wall; or
  - A combination of both.
- In accordance with Sections 4.6.4 and C.4.6.4 of the *CHBDC (2006)* and its *Commentary (2006)*, for structures which do not allow lateral yielding (i.e. the abutment walls for this structure), the horizontal seismic coefficient, k_h, used in the calculation of the seismic lateral earth pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0.098). For structures which allow lateral yielding (i.e. the wing walls for this structure), k_h is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.033). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.
- The following seismic active pressure coefficients (K_{AE}) and seismic passive pressure coefficients (K_{PE}) for two backfill cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} and the minimum K_{PE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground

surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

Wall Type	Case I	Case II
Yielding wall	0.38	0.26
Non-yielding wall	0.43	0.31

SEISMIC PASSIVE PRESSURE COEFFICIENTS, K_{PE}

Wall Type	Case I	Case II
Yielding wall	4.2	7.17
Non-yielding wall	3.94	6.76

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.065. This corresponds to displacements of up to approximately 16 mm at this site.
- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K\,\gamma'\,d + (K_{AE} - K)\,\gamma'\,(H-d)$$

where $\sigma_h(d)$ is the lateral earth pressure at depth d, (kPa)

K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);

K_{AE} is the seismic active earth pressure coefficient;

γ' is the effective unit weight of the backfill soil (kN/m^3), taken as soil unit weight given above;

d is the depth below the top of the wall (m); and

H is the total height of the wall above the toe (m).

The K_{AE} values for the condition of flat ground surface behind the wall include the effect of wall friction (δ $=\emptyset'/2$) of this site.

5.10 Scour Protection and Frost Protection

It is understood that the sheet pile walls were installed on the west side of the existing abutment for scour protection. The sheet pile walls were installed many years after the bridge construction to prevent further erosion at the east abutment. A core hole was attempted on the concrete platform between the sheet pile wall and the abutment wall. The corehole encountered refusal due to the presence of steel rebar between the sheet piles and abutment. This rebar is a potential obstruction and may need to be removed to facilitate either the abutment replacement or underpinning.

Abutment stems, pier cap, and for any associated concrete wing walls/retaining walls, should be founded at a minimum depth of 1.8 m below the lowest surrounding grade, to provide adequate protection against frost penetration. The bridge approach slopes and slopes at the abutments except for the section with existing sheet piles should be armoured with at least 1.0 m of stone riprap and rock blocks. It should be noted that the riprap or rock blocks should not be counted as earth cover for frost protection.

Rigid insulation may be used as an alternative to providing the standard depth of soil cover for frost protection of exterior footings. Assuming a minimum depth of soil cover of 300 mm adjacent to the exterior foundation wall, the insulation should consist of a 100 mm (4 inch) thick layer of Styrofoam HI-40 insulation (or equivalent), extending 1.8 m laterally (with a 2 percent downward slope) from the intersection of the exterior foundation wall and the top of footing. A 100 mm thick vertical section of Styrofoam should be placed against the upper foundation wall up to exterior grade level.

In addition, the bearing soil or bedrock and fresh concrete should be protected from freezing during cold weather construction.

5.11 Existing Abutment Wall and Wing Wall

As noted above, separation along the construction joint was observed in the concrete core drilled through the abutment wall. This construction joint should be repaired to restore the bonding between the concrete above and below the construction joint.

The spalling and delamination also observed below the construction joint should be repaired if the underpinning option is selected.

Stability analyses of the existing abutment and wing walls should be carried out if underpinning is selected. The sliding and overturning stability analyses should be carried out by the structure engineer and the global stability should assessed by the geotechnical engineer once the underpinning design is available.

The following parameters can be used in the stability analyses for the concrete wall structures:

- Unit weight of existing granular backfill = γ = 20 kN/m³
- Unit weight of native clayey silt = γ = 17 kN/m³
- Unit weight of concrete walls = γ = 21.5 kN/m³
- Unit weight of silty sand to sandy silt fill = γ = 18 kN/m³
- Unit weight of organic fill = γ = 17 kN/m³
- Unit weight of water = γ_w = 9.8 kN/m³
- "Active" lateral earth pressure coefficient = K_a = 0.3
- "At Rest" lateral earth pressure coefficient = K_o = 0.5
- Unfactored coefficient of friction between unboneded concrete and concrete = μ = 0.7

A groundwater water level of 1.9 m below the ground surface may be assumed for the stability analyses. The groundwater level could be assumed to be at the elevation of new subdrains which could be installed during the replacement of the granular backfill behind the walls.

5.12 Monitoring Program

Although a formal monitoring program has not been established to measure the observed movement, from our discussions with the swing bridge operator, it is understood that the rate of relative movement between the swing bridge and the fixed bridge has increased, requiring more frequent adjustment of the wood bridge deck. If the bridge repair cannot be scheduled in the short term, it is recommended that an instrumentation monitoring program be established to measure the magnitude and rate of future movement so that the structural engineer in consultation with Parks Canada can make an informed decision regarding the continued safe operation of the bridge.

5.13 Construction Considerations

5.13.1 Excavation and Groundwater Control

The excavations for proposed abutment replacement may extend to a maximum depth of about 3.5 m (i.e. the depth of the existing abutment wall) below the ground surface through the existing fill. If space permits, open-cut excavations to the proposed depths should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The existing fills are classified as Type 3 soil according to the OHSA; the existing very soft to soft silty clay and loose sandy silt to silty sand deposits under groundwater table are classified as Type 4 soil according to OHSA; temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1 horizontal to 1 vertical for Type 3 soil and 3 horizontal to 1 vertical for Type 4 soil. In addition, care must be taken during excavation to ensure that adequate support is provided for any existing structures and underground services located adjacent to the excavations.

If adjacent structures and/or utilities are susceptible to damage from construction induced settlement, then a more positive excavation support such as sheet pile or strutted soldier pile and lagging wall may be considered. However, the presence of the cobbles and boulders may make present difficulties for driving sheet piling or soldier piles.

Groundwater control at these locations would be required to allow for construction of foundation elements in a dry condition. It should be noted that wet cohesionless sandy silt/silty sand and silt were encountered below the very soft to soft silty clay. Depending upon the actual thickness and extent of these wet sandy silt/silty sand/silt zones, some form of positive groundwater control may be required to maintain the stability of the base and side slopes of the excavations in these areas, in addition to pumping from sumps.

A cofferdam or similar structure may be necessary to provide temporary excavation support and to facilitate groundwater control within the excavation. Groundwater control measures or dewatering should be implemented by a specialist contractor and carried out to a depth of at least 0.6 m below the excavation base level, or as necessary to ensure stable conditions during excavation. It should be noted that a complete cut-off may not be achieved by sheet pile walls, and seals on both sides of the walls may be needed.

The underside of the existing abutment is about 3.5 m below the ground surface. It should be noted that the native very soft to soft and very loose to loose sandy/silty deposits below the groundwater table are very easily disturbed and may not be able to support heavy construction equipment. Concrete mud slabs or granular pads should be considered to provide stable working surfaces for the construction equipment.

Pumping discharges should conform to the Ministry of Environments guidelines as well as any requirements from the local municipality, conservation agencies and Parks Canada.

It would be prudent to carry out a "public digging" (i.e. test pitting) during the tender stage, to allow prospective bidders to assess the subsurface conditions and determine the type of groundwater control required, consistent with their equipment capabilities and the actual groundwater conditions at that time. The locations of the test pits should be determined in consultation with the geotechnical engineer. It should be noted that groundwater control measures that extract more than 50,000 L/day of water are subject to a Permit to Take Water (PTTW), as regulated by the MOE. Additional hydrogeological study may be warranted in support of the PTTW depending on construction methods and equipment used.

5.13.2 Cobbles and Boulders

Boulders and cobbles are commonly encountered in the glacially derived soils/tills of southern Ontario. The specific presence of boulders can significantly affect the selection of equipment and progress of construction works, especially in pile driving or boring. The presence of such obstructions may also affect the excavation works and the installation of piles (depending on the pile cap level) if adopted for foundation design. The presence of the cobbles and boulders are inferred from observations during the drilling (i.e. practical refusal to further augering, auger grinding, etc). It should be noted that the size and quantity of the cobbles and boulders within the soil deposits are not able to be determined. It is recommended that this should be included in the Contract Document to warn the Contractor of these obstructions and to ensure that the Contractor is equipped to handle such obstruction.

5.14 Approach Embankment Design

A short section of the approach embankment intersects Hartley Road east of the east abutment. The existing bridge approach embankment along this section of road is about 2 m to 3 m high. Based on visual observations, the existing east approach embankment slopes appear to be stable. Analyses of the slope stability and settlement of the embankment would only be recommended if a significant grade change or widening is planned for the approach embankment.

5.14.1 Subgrade Preparation and Backfill Behind the Abutment Wall and Wing Walls

5.14.1.1 Removal of Existing Backfills

Based on the gradation analysis results, the backfill materials (i.e. sandy silt fill and organic fill) are considered to be highly frost susceptible. There is no evidence of a drainage system behind the abutment wall and wing walls. It is expected that the existing backfill will be removed for either the full replacement or underpinning options. The excavation should be backfilled to the underside of the pavement structure using Granular B, Type I material. The Granular B backfill should be placed in accordance with Special Provision SP206S03. The final lift prior to placement of the granular subbase and base courses should be compacted to at least 100 per cent of the standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified geotechnical personnel during all engineered fill placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Depending on the source and gradation of the Granular B backfill, the unit weight of the replacement backfill may be greater than the unit weight of the original backfill. An increase in the unit weight of the backfill material would increase the loading on the native soils and could potentially result in consolidation of the very soft to soft cohesive soils at the abutment. The potential consolidation should be evaluated once the unit weight of the backfill is available to determine the magnitude of the estimated embankment settlement and the associated down drag forces acting on the pile foundations. The use of a lightweight fill may be considered for part of the backfill zone to balance the backfill loading and avoid consolidation, if required. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00. The drainage system should be installed above the local groundwater level.

The abutment wall and wing walls should be properly braced to provide sufficient temporary support during construction. The excavation and brace plans should be submitted and approved by the engineer. The requirements for temporarily supporting the existing bridge during construction should be evaluated by the structure engineer.

To reduce surface water erosion on the embankment side slopes, topsoil and seeding or pegged sod should be placed as soon as possible in accordance with OPSS 572. If this protection is not in place before winter, then alternate protection measures, such as covering the slope with straw or gravel sheeting, is recommended to reduce the potential for remedial works being required on the side slopes in the Spring prior to topsoil and seeding.

5.15 Pavement Design

Based on the existing soil and pavement conditions, the following pavement design is recommended for the restoration of the pavement on the approach embankment. The pavement design provided in the following table should be reviewed once traffic loading conditions, including heavy truck traffic volumes, are known to verify that the pavement designs are sufficient for the project requirements.

MATERIAL		THICKNESS OF PAVEMENT ELEMENTS (mm)
Asphaltic Material (OPSS 1150)	HL 3	40
	HL 8	100 (two lifts)
Granular Material (OPSS 1010)	Granular A Base	150
	Granular B, Type I Subbase	500
		Prepared and Approved Subgrade

Prior to placing the granular subbase material, the exposed soil subgrade should be heavily proofrolled in conjunction with an inspection by qualified geotechnical personnel. Remedial work (i.e. further subexcavation and replacement) should be carried out on any disturbed, softened or poorly performing zones, as directed by geotechnical personnel.

The granular subbase and base materials should be uniformly compacted to 100 percent of their standard Proctor maximum dry densities. The asphalt materials should be compacted to 92 to 96.5 percent of their Marshall Relative Densities (MRD), as measured in the field using a nuclear density gauge.

In addition, in order to preserve the integrity of the pavement, continuous subdrains should be placed along both sides of the road. The invert of the subdrains should be at least 300 mm below the bottom of the Granular B subbase and should be sloped to drain to the catchbasins. The subdrains should consist of perforated pipe wrapped in a suitable geotextile and surrounded on all sides with a minimum thickness of 150 mm of clean free draining sand such as concrete sand.

The above pavement designs should provide serviceable pavements for the anticipated traffic levels over a normal design period of fifteen years, provided that timely maintenance is carried out (i.e. crack sealing).

Where new pavement abuts existing pavement (e.g. at the construction limits), proper longitudinal lap joints should be constructed to key the new asphalt into the existing pavement. The existing asphalt edges should be provided with a proper sawcut edge prior to keying in the new asphalt. It should be ensured that any undermined or broken edges resulting from the construction activities are removed by the sawcut.

6.0 CLOSURE

Prior to finalizing the design of the proposed bridge and associated works, the geotechnical aspects of the design drawings/specifications should be reviewed by this office to confirm that the intent of this report has been met. Further, prior to tendering, the geotechnical aspects of the final design drawings/specifications and proposed construction methodology should be reviewed by this office to confirm that the intent of this report has been met.

During construction, Golder personnel should confirm that the subsurface conditions encountered at the foundation location are consistent with those in the boreholes. Sufficient site visits should also be carried out during bridge construction to monitor conformance with the pertinent project specifications.

We trust that this report provides sufficient geotechnical engineering information to complete the design of this project. If you have any questions regarding the contents of this report or require additional information, please do not hesitate to contact this office.

Yours truly,

GOLDER ASSOCIATES LTD.

LICENSED PROFESSIONAL ENGINEER
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http://capws/sites/capws2/p11110118hamletbridge/reports/11-1111-0118 rep 2012'03'07 geotechnical investigation final.docx

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.).

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60° conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	c_u, s_u kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_S)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π = 3.1416
ln x, natural logarithm of x
$\log_{10}$ x or log x, logarithm of x to base 10
g acceleration due to gravity
t time
F factor of safety
V volume
W weight

II. STRESS AND STRAIN

γ shear strain
Δ change in, e.g. in stress: $\Delta\ \sigma$
ε linear strain
ε_v volumetric strain
η coefficient of viscosity
ν Poisson's ratio
σ total stress
σ' effective stress ($\sigma' = \sigma - u$)
σ'_{vo} initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$ principal stresses (major, intermediate, minor)
σ_{oct} mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ shear stress
u porewater pressure
E modulus of deformation
G shear modulus of deformation
K bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$ bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$ dry density (dry unit weight)
$\rho_w(\gamma_w)$ density (unit weight) of water
$\rho_s(\gamma_s)$ density (unit weight) of solid particles
γ' unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R relative density (specific gravity)of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e void ratio
n porosity
S degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

(a) Index Properties (con't.)

w water content
w_l liquid limit
w_p plastic limit
I_P plasticity Index = $(w_l - w_p)$
w_S shrinkage limit
I_L liquidity index = $(w - w_p) / I_p$
I_C consistency index = $(w_l - w) / I_p$
e_{max} void ratio in loosest state
e_{min} void ratio in densest state
I_D density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(c) Hydraulic Properties

h hydraulic head or potential
q rate of flow
v velocity of flow
i hydraulic gradient
k hydraulic conductivity (coefficient of permeability)
j seepage force per unit volume

(d) Consolidation (one-dimensional)

C_c compression index (normally consolidated range)
C_r recompression index (overconsolidated range)
C_S swelling index
C_α coefficient of secondary consolidation
m_v coefficient of volume change
c_v coefficient of consolidation
T_v time factor (vertical direction)
U degree of consolidation
σ'_p pre-consolidation pressure
OCR Overconsolidation ratio $= \sigma'_p / \sigma'_{vo}$

(e) Shear Strength

τ_p, τ_r peak and residual shear strength
ϕ' effective angle of internal friction
δ angle of interface friction
μ coefficient of friction = $\tan \delta$
c' effective cohesion
c_u, S_u undrained shear strength ($\phi = 0$ analysis)
p mean total stress $(\sigma_1 + \sigma_3)/2$
p' mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q $(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u compressive strength $(\sigma_1 - \sigma_3)$
S_t sensitivity

Notes: 1. $\tau = c' + \sigma' \tan \phi'$
2. Shear strength = (Compressive strength)/2

PROJECT: 11-1111-0118

LOCATION: SEE FIGURE 2

RECORD OF BOREHOLE: BH 1

BORING DATE: September 28, 2011

SHEET 1 OF 1

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: TRACK MOUNTED POWER AUGER AUTOMATIC HAMMER, 150 mm Diameter Solid Stem Augers

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ — 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢—○W—⊣ Wl — 10 20 30 40

SOIL PROFILE – DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
GROUND SURFACE					
ASPHALT (80 mm)	0.00				
GRANULAR BASE	0.08	1A	50 DO	11	MH
Compact dark brown silty sand, trace to some gravel (FILL)	0.35	1B			
Loose dark brown and brown mottled fine sandy silt, trace clay, trace fine sand, trace organic (FILL)	0.76	2	50 DO	4	
Loose dark brown organic silt, some sand, trace clay (FILL)	1.37	3	50 DO	3	
Firm brown and grey mottled CLAYEY SILT to SILTY CLAY, some sand	2.13	4	50 DO	5	
		5	50 DO	5	
Very loose dark grey ORGANIC SILT, trace to some clay, trace sand, trace rootlets	4.04	6	50 DO	W/H	
Very loose dark grey SILTY FINE SAND, trace clay	5.56	7	50 DO	4	
Very dense grey SILTY SAND and GRAVEL, organic stains, containing cobbles and boulders	7.09	8	50 DO	50/.1	
END OF BOREHOLE DUE TO AUGER REFUSAL ON BIOTITE GNEISS BECROCK	7.87				

PIEZOMETER OR STANDPIPE INSTALLATION:

Water encountered during drilling at a depth of 4.5 m below ground surface, Sept. 28/11

Water level in open borehole at a depth of 4.1 m below ground surface upon completion of drilling, Sept. 28, 2011

DEPTH SCALE 1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

PROJECT: 11-1111-0118

RECORD OF BOREHOLE: BH 2

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

BORING METHOD: TRUCK MOUNTED POWER AUGER, 150 mm Diameter Solid Stem Augers (to 7.87 m)

DEPTH (m)	DESCRIPTION	ELEV. DEPTH (m)
	GROUND SURFACE	
0	ASPHALT (85 mm)	0.00
	GRANULAR BASE (Slight hydrocarbon-like odour)	0.09
	Brown silty sand and gravel (Slight hydrocarbon-like odour) (FILL)	0.40
	Very loose brown fine sandy silt, trace gravel (FILL)	0.76
	Loose dark brown organic silt, some sand, trace clay, trace roots (FILL)	1.37
	Weathered concrete (PROBABLE ABUTMENT FOOTING)	2.40
	Very soft to soft grey SILTY CLAY to CLAYEY SILT, trace sand, organic stains	3.40
	Very loose dark grey FINE SANDY SILT, trace clay, organic stains, trace decayed root fragments	5.26
	Very loose grey SILTY FINE SAND, organic stains	5.74
	Compact grey SAND, some gravel, some silt, organic stains	6.40
	Continue with coring Fresh to slightly weathered dark grey to black fine to medium grained BIOTITE GNEISS BEDROCK	7.87
	END OF BOREHOLE T.C.R. (Total Core Recovery) = 98% S.C.R. (Solid Core Recovery) = 97%	9.40

SAMPLE NUMBER	TYPE	BLOWS/0.3m	ADDITIONAL LAB. TESTING
1	AS	-	
2	50 DO	4	MH
3	50 DO	5	MH
4A			
4B	50 DO	11/.1	
5	50 DO	50/.03	
6	50 DO	2	MH
7	50 DO	-	
8A			
8B	50 DO	1	
9	50 DO	13	MH

DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m: 20 40 60 80

SHEAR STRENGTH Cu, kPa: nat V. + Q - ● rem V. ⊕ U - ○ : 20 40 60 80

HYDRAULIC CONDUCTIVITY, k, cm/s: 10^{-6} 10^{-5} 10^{-4} 10^{-3}

WATER CONTENT PERCENT: Wp ⊢ ○W ⊣ Wl : 10 20 30 40

PIEZOMETER OR STANDPIPE INSTALLATION: Bentonite Seal; Silica Sand Filter; Bentonite Seal

Water encountered during drilling at a depth of 4.5 m below ground surface, Sept. 28/11

CONTINUED NEXT PAGE

RECORD OF BOREHOLE: BH 2

PROJECT: 11-1111-0118

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES: NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp ⊢—○W—⊣ Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		— CONTINUED FROM PREVIOUS PAGE —									
10		R.Q.D. (Rock Quality Designation) = 93%									Water measured in piezometer at a depth of 1.9 mbelwo ground surface upon completion of drilling, Oct. 22/11
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

RECORD OF BOREHOLE: BH 3

PROJECT: 11-1111-0118

LOCATION: SEE FIGURE 2

BORING DATE: September 28, 2011

SHEET 1 OF 1

DATUM: Geodetic

SAMPLER HAMMER, 64kg; DROP, 760mm

PENETRATION TEST HAMMER, 64kg; DROP, 760mm

DEPTH SCALE METRES	BORING METHOD	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLES NUMBER	TYPE	BLOWS/0.3m	DYNAMIC PENETRATION RESISTANCE, BLOWS/0.3m (20 40 60 80); SHEAR STRENGTH Cu, kPa nat V. + Q - ● rem V. ⊕ U - O (20 40 60 80)	HYDRAULIC CONDUCTIVITY, k, cm/s (10^{-6} 10^{-5} 10^{-4} 10^{-3}); WATER CONTENT PERCENT Wp —O W— Wl (10 20 30 40)	ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
		GROUND SURFACE									
0	Coring	CONCRETE (NORTH ABUTMENT)		0.00							
		Brown to grey CLAYEY SILT, some sand, organic stains, trace rootlets, zones of silty fine sand		3.48	1		-			MH	
					2		-			MH	
4		END OF BOREHOLE		4.04							

Depth scale: 0, 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: AZ

CHECKED: DBL

GTA-BHS 001 11-1111-0118.GPJ GAL-MIS.GDT 11/9/11 MK Sept. 2011

PROJECT

DELCAN
HAMLET BRIDGE (FIXED SPAN)
HAMLET, ONTARIO

TITLE

KEY PLAN

PROJECT NO. 11-1111-0118		
DESIGN	SS	OCT. 2011
GIS	SS/JT	OCT. 2011
CHECK	DBL	Mar. 2012
REVIEW		

SCALE AS SHOWN | REV. 0.0

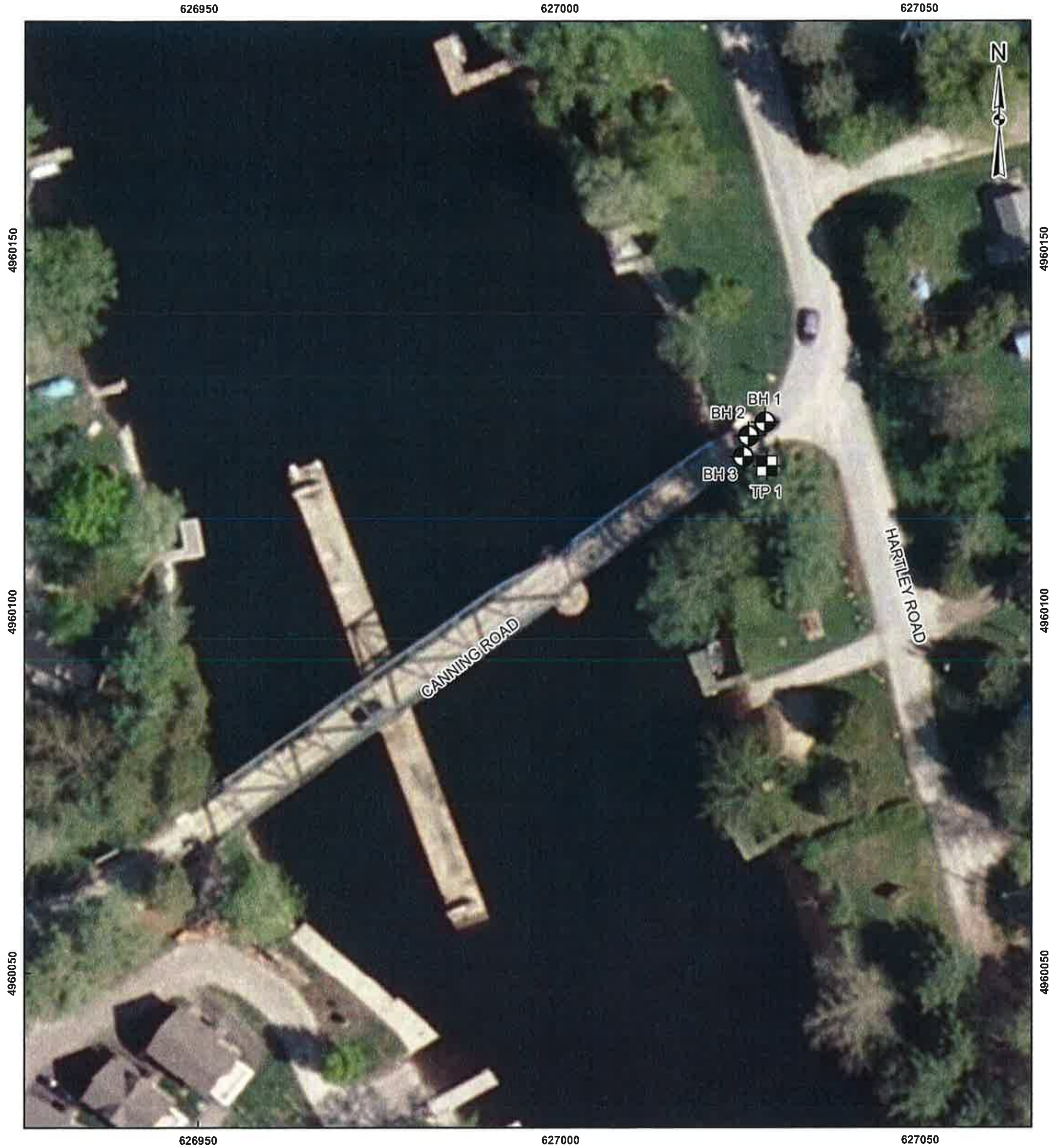
FIGURE 1

Golder Associates
Whitby, Ontario

REFERENCE

Base Data - MNR NRVIS, obtained 2004, CANMAP v2006.4
Produced by Golder Associates Ltd under licence from
Ontario Ministry of Natural Resources, © Queens Printer 2008
Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 17

G:\Projects\2011\11-1111-0118_Hamlet_Ontario\GIS\MXDs\Reporting\1111110118A001.mxd



LEGEND

LOCATION OF BOREHOLE

LOCATION OF TEST PIT

15 7.5 0 15 30

SCALE 1:750 METRES

PROJECT

DELCAN
HAMLET BRIDGE (FIXED SPAN)
HAMLET, ONTARIO

TITLE

BOREHOLE AND TEST PIT LOCATION PLAN

REFERENCE

Imagery - Bing Maps © 2009 Microsoft Corporation and its data suppliers
Produced by Golder Associates Ltd under licence from
Ontario Ministry of Natural Resources, © Queens Printer 2008
Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 17N

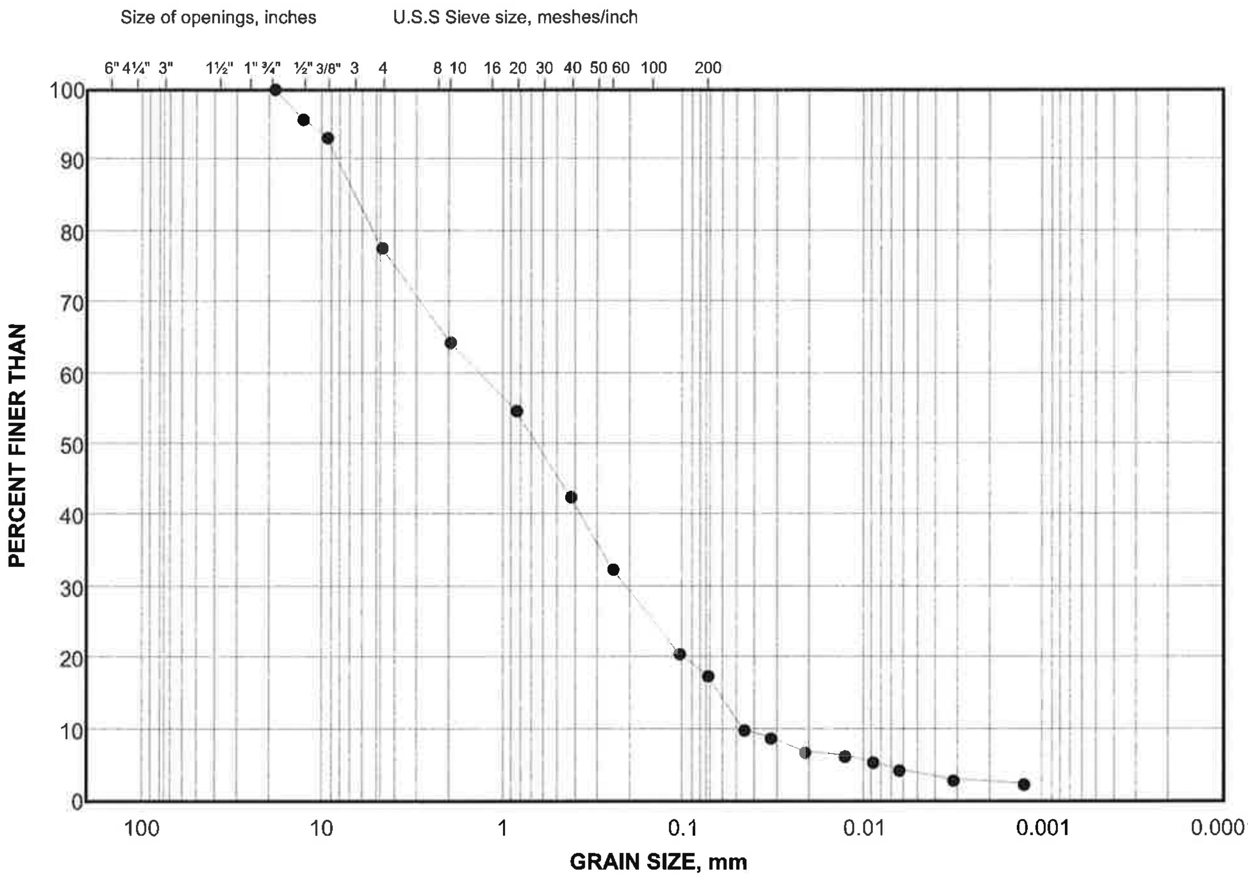
Golder Associates
Whitby, Ontario

PROJECT NO. 11-1111-0118			SCALE AS SHOWN	REV. 0.0
DESIGN	SS	OCT. 2011	**FIGURE 2**	
GIS	SS/JT	OCT. 2011		
CHECK	DBL	Mar. 2012		
REVIEW				

G:\Projects\2011\11-1111-0118_Hamlet_Ontario\GIS\MXDs\Reporting\1111110118AA02.mxd

GRAIN SIZE DISTRIBUTION
SILTY SAND (FILL)

FIGURE 3



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	1	1B	0.35 - 0.76

Project Number: 11-1111-0118

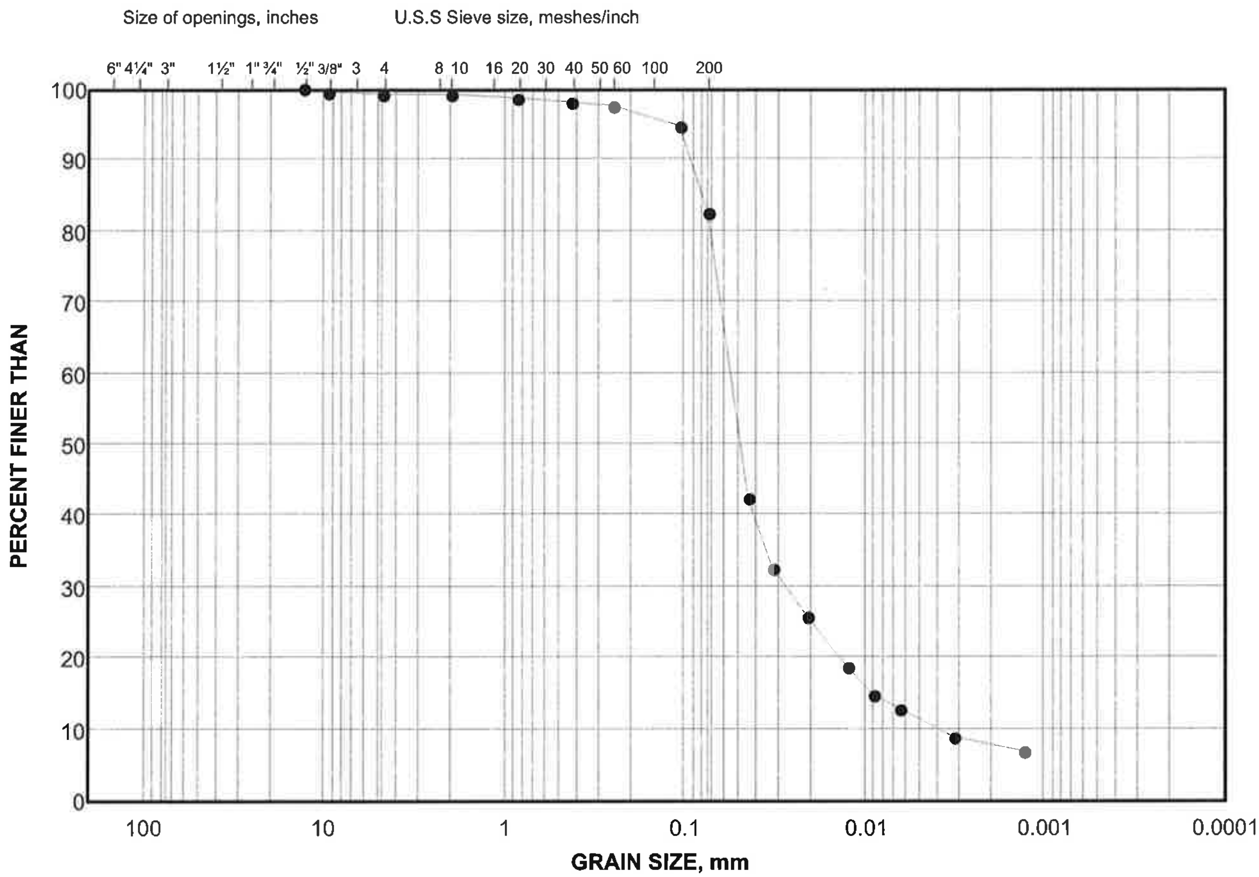
Checked By: DBL

Golder Associates

Date: 28-Oct-11

GRAIN SIZE DISTRIBUTION
SANDY SILT (FILL)

FIGURE 4



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	2	0.76 - 1.22

Project Number: 11-1111-0118

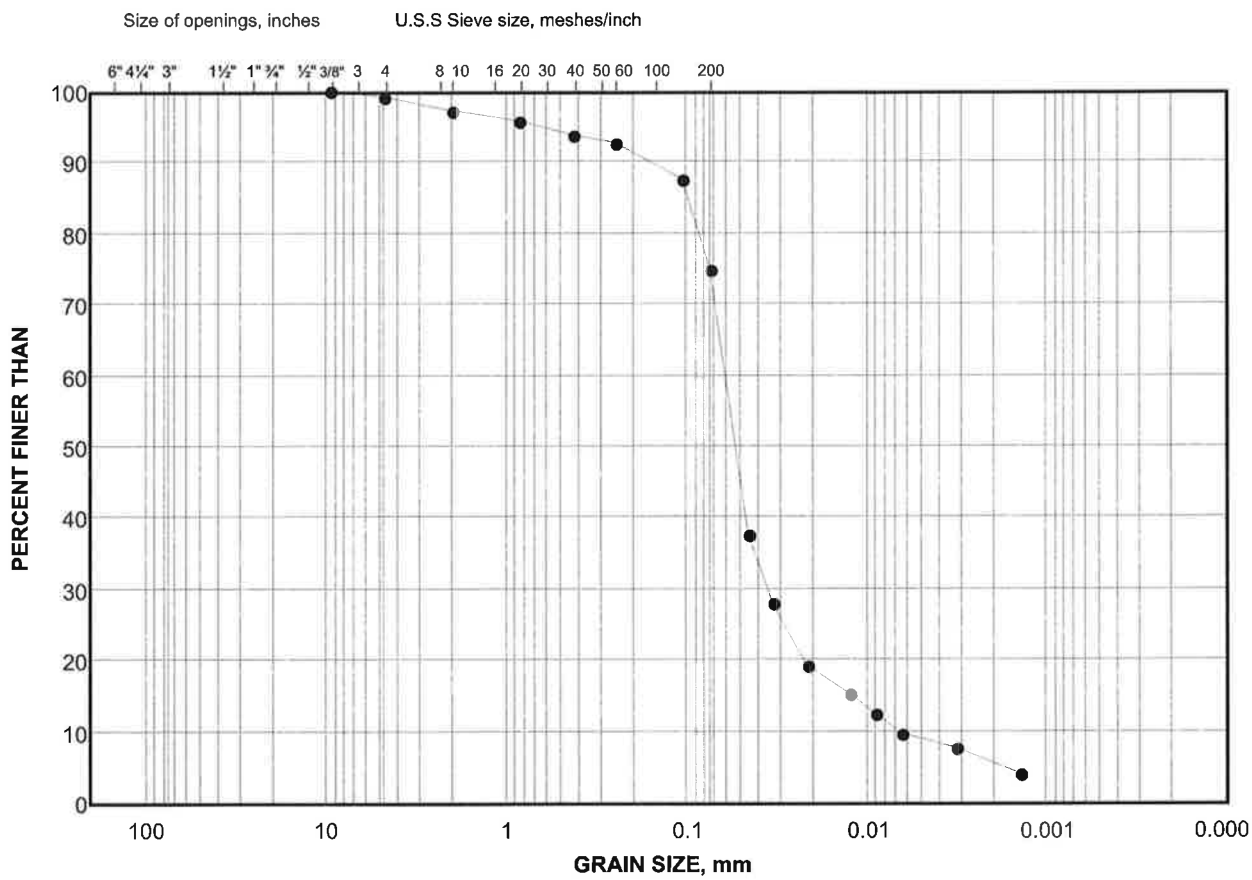
Checked By: DBL

Golder Associates

Date: 28-Oct-11

GRAIN SIZE DISTRIBUTION
ORGANIC SILT (FILL)

FIGURE 5



COBBLE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY SIZES
SIZE	GRAVEL SIZE		SAND SIZE			FINE GRAINED

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	3	1.52 - 1.98

Project Number: 11-1111-0118

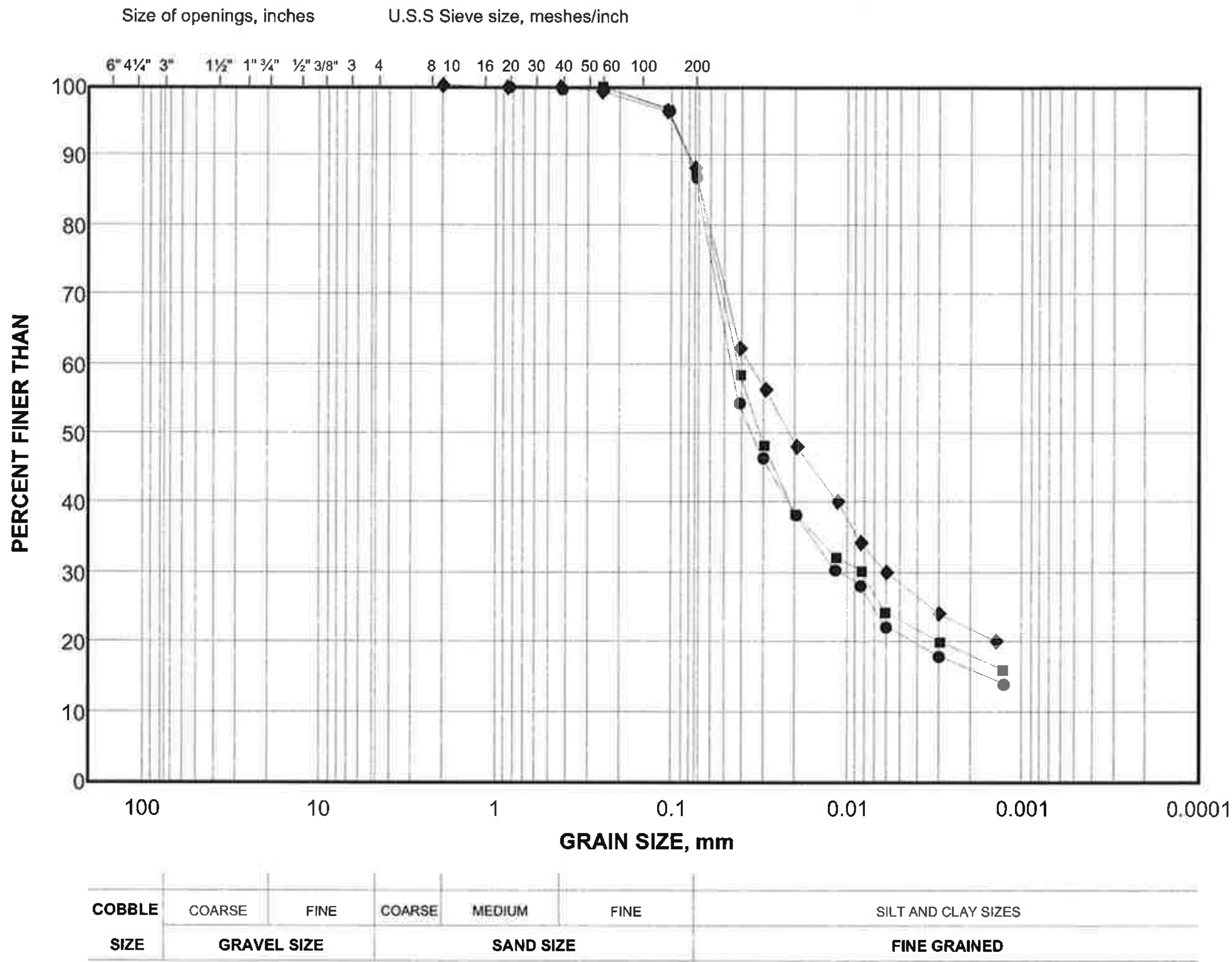
Checked By: DBL

Golder Associates

Date: 07-Nov-11

GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIGURE 6



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	3	1	3.48 - 3.73
■	3	2	3.73 - 4.04
◆	2	6	4.57 - 5.03

Project Number: 11-1111-0118

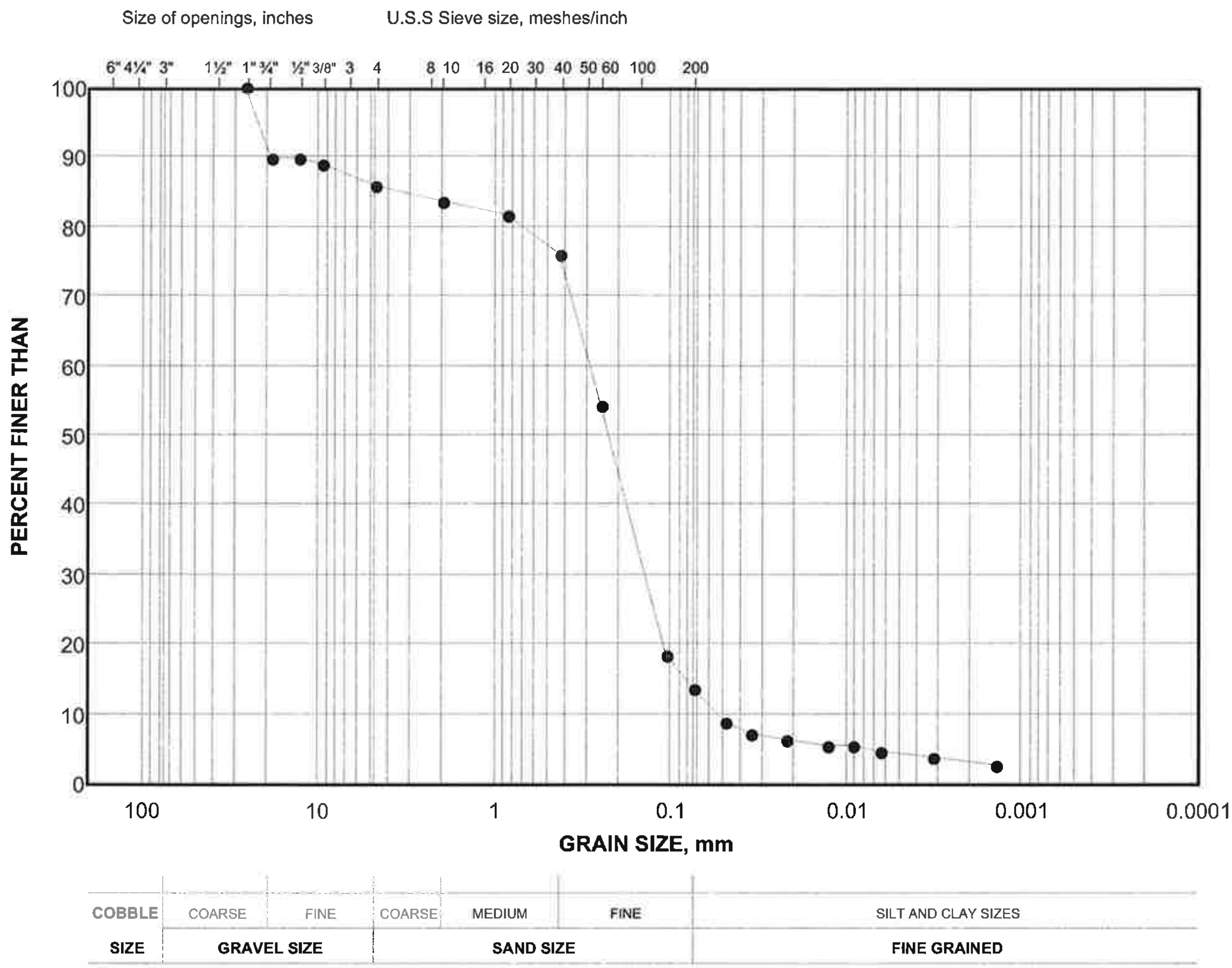
Checked By: DBL

Golder Associates

Date: 07-Nov-11

GRAIN SIZE DISTRIBUTION
SAND

FIGURE 7



LEGEND

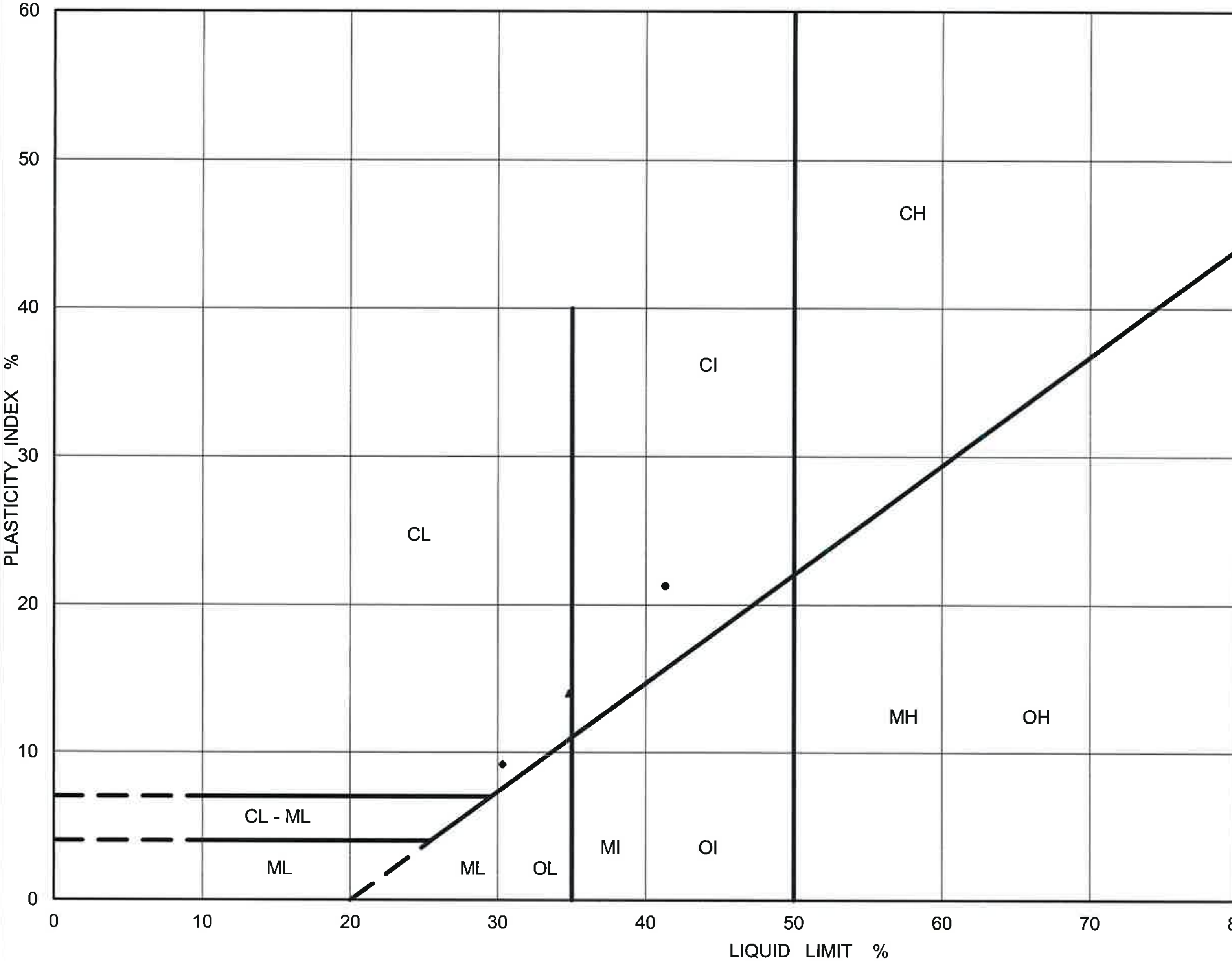
SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	2	9	7.62 - 8.08

Project Number: 11-1111-0118

Checked By: DBL

Golder Associates

Date: 07-Nov-11



BH	SAMPLE	SYMBOL
2	6	●
3	1	◆
3	2	▲

Golder Associates

PLASTICITY CHART

Figure No.	8
Project No.	11-1111-0118
Checked By:	DBL

OBTAINING AND TESTING DRILLED CORES FOR COMPRESSIVE STRENGTH TEST
(CSA A23.2-14C)

October 24, 2011

Golder Project Number: 11-1111-0118

Figure 9

Golder Associates Ltd.
100 Scotia Court
Whitby, ON L1N 8Y6

ATTENTION: Mr. David Lui

PROJECT: **Hamlet Bridge**

Date Received: October 13, 2011 **Date Tested:** October 18, 2011

Core Number:	**C2**	**C2**	**BH2 - Bedrock**
Depth:	1'4" – 3'5"	9'5" – 10'1"	26'4" – 27'4"
Golder Lab Number:	C-11-1355	C-11-1356	C-11-1357
Moisture Condition at Time of Test	Wet	Wet	Wet
Capping Material	Sulphur	Sulphur	Sulphur
Average Diameter, (mm)	94.1	94.2	93.0
Average Length (mm)	172.2	145.4	1.98
Density, (Mg/m^3)	2.255	2.165	2.434
Load, (kN)	130.24	102.47	237.07
Compressive Strength, (MPa)	18.7	14.7	136.7
Corrected Compressive Strength, (MPa)	**18.4**	**14.2**	**136.4**

Reviewed by: ______________________
Annmarie Jarvis, Laboratory Supervisor

Figure 10A

UNCONFINED COMPRESSION TEST (UC)

ASTM D 2166 - 06

SAMPLE IDENTIFICATION

PROJECT NUMBER	11-1111-0118	SAMPLE NUMBER	2
BOREHOLE NUMBER	3	SAMPLE DEPTH, m	3.73-4.04

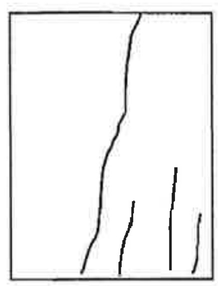
TEST CONDITIONS

MACHINE SPEED, mm/min	0.76	TYPE OF SPECIMEN	Thin wall tube sample
RATE OF AXIAL STRAIN, %/min	0.75	L/D	2.02

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.10	WATER CONTENT, (specimen) %	25.53
SAMPLE DIAMETER, cm	4.99	UNIT WEIGHT, kN/m^3	19.56
SAMPLE AREA, cm^2	19.58	DRY UNIT WT., kN/m^3	15.58
SAMPLE VOLUME, cm^3	197.76	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	394.58	VOID RATIO	0.70
DRY WEIGHT, g	314.33		

FAILURE SKETCH



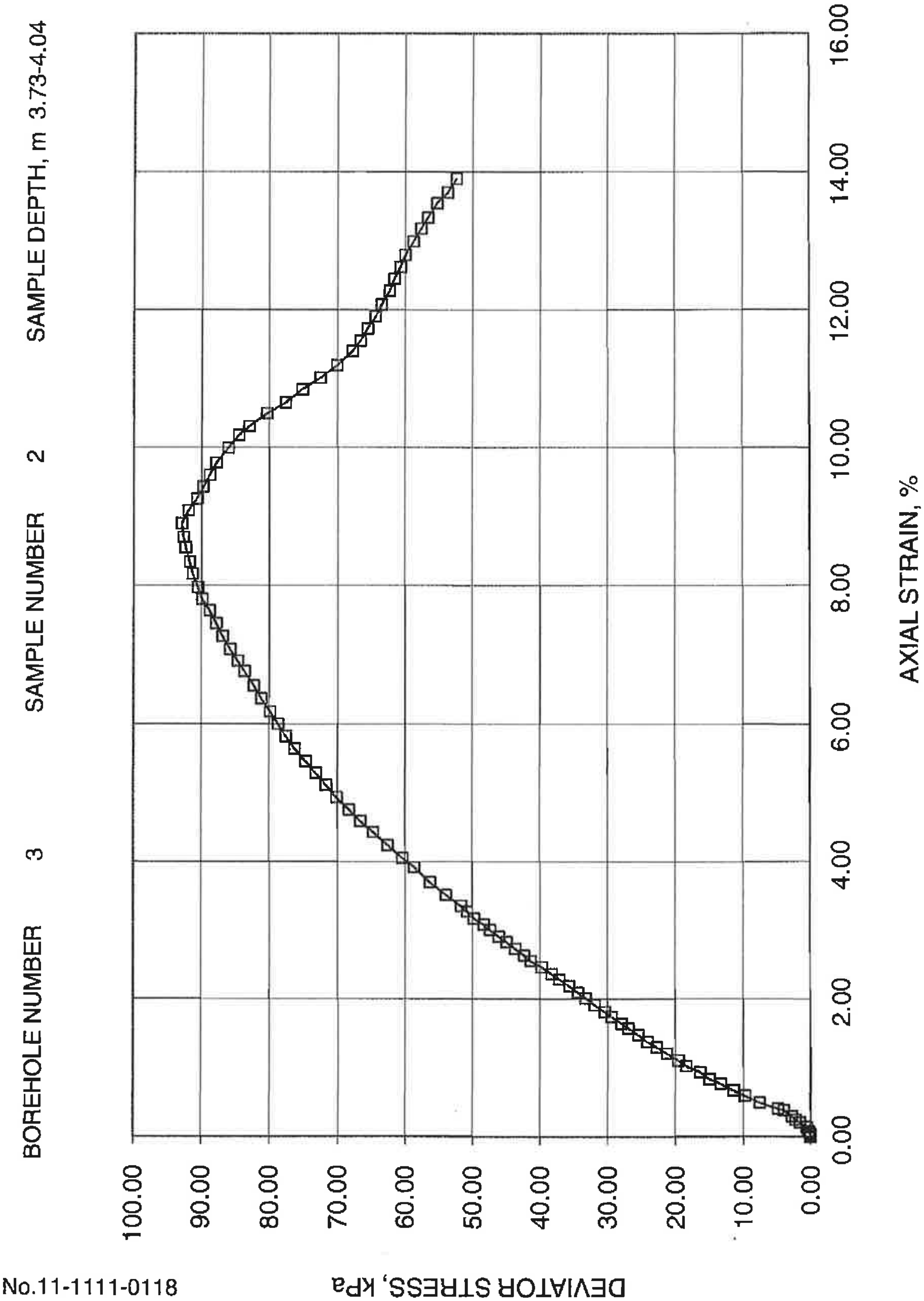
TEST RESULTS

STRAIN AT FAILURE, %	8.9	COMPRESSIVE STRESS, kPa	93

REMARKS:

DATE: 10/12/2011

UNCONFINED COMPRESSION TEST (UC)	FIGURE 10B



Project No.11-1111-0118

DENSITY AND POROSITY DETERMINATIONS OF IRREGULAR SHAPE SAMPLES

ASTM D 7263 - 09 Method A

Borehole Number	3 (A)	3 (B)
Sample Number	1	1
Depth, m	3.5-3.8	3.5-3.8
Wet Mass of Soil in Air, g	433.25	455.90
Wet Mass of Soil + Wax in Air, g	456.30	476.60
Wet Mass of Soil + Wax in Water, g	210.90	222.40
Weight of Wax, g	23.05	20.70
Displaced Volume, cm^3	245.40	254.20
Displaced Wax, cm^3	25.39	22.80
Volume of Soil, cm^3	220.01	231.40
Specific Gravity, assumed	2.70	2.70
Volume of Solids, cm^3	125.26	132.23
Volume of Voids, cm^3	94.75	99.18
Porosity	0.43	0.43
Water Content, %	28.10	27.70
Unit Weight, kN/m^3	19.31	19.32
Dry Unit Weight, kN/m^3	15.08	15.13

Project Number	11-1111-0118	Tested By	Larry
Date Tested	10/7/2011	Checked By	AH

Figure 11B

DENSITY (UNIT WEIGHT) OF SOIL SPECIMENS

ASTM D 7263 Method B

Borehole Number	2
Sample Number	7
Sample Depth, m	4.6-5.2
Weight of Soil + Tube, g	179.77
Weight of Tube, g	76.43
Weight of Soil, g	103.34
Diameter of Sample, cm	6.33
Length of Sample, cm	1.90
Volume of Sample, cc	59.71
Water Content, %	48.1
Wet Density, Mg/m^3	1.731
Dry Density, Mg/m^3	1.169
Unit Weight, kN/m^3	16.97
Borehole Number	
Sample Number	
Sample Depth, m	
Weight of Soil + Tube, g	
Weight of Tube, g	
Weight of Soil, g	
Diameter of Sample, cm	
Length of Sample, cm	
Volume of Sample, cc	
Water Content, %	
Wet Density, Mg/m^3	
Dry Density, Mg/m^3	
Unit Weight, kN/m^3	

Project Number	11-1111-0118	Tested By	Lina
Date Tested	10/25/2011	Checked By	MM

Golder Associates

Figure 12

ORGANIC CONTENT (BURNING METHOD)

BOREHOLE NUMBER		2					
SAMPLE NUMBER		3					
CRUCIBLE NUMBER			8				
WEIGHT OF CRUCIBLE, g	W1	28.95	28.84				
WEIGHT OF CRUCIBLE & AIR DRY SAMPLE, g	W2	57.67	55.49				
WEIGHT OF AIR DRY SAMPLE (ORIGINAL), g	W2-W1	28.72	26.65				
WEIGHT AFTER BURNING SOIL & CRUCIBLE, g	W3	56.94	54.78				
WEIGHT OF ORGANICS, g	W2-W3	0.73	0.71				
PERCENT OF ORGANICS, %	((W2-W3)/(W2-W1))x100	2.54	2.66				
ORGANIC CONTENT, %		2.6					

PROJECT NUMBER	11-1111-0118	DATE OF TESTING	10/25/2011
TESTED BY	Renato / Lina	CHECKED BY	

Notes:

1. Samples dried at 110 degree centigrade prior to testing.

2. Test performed according to ASTM D2974 Standard, test method C.

3. Organic matter determined by burning the oven dried samples in a muffle furnace at 440 degree centigrade.

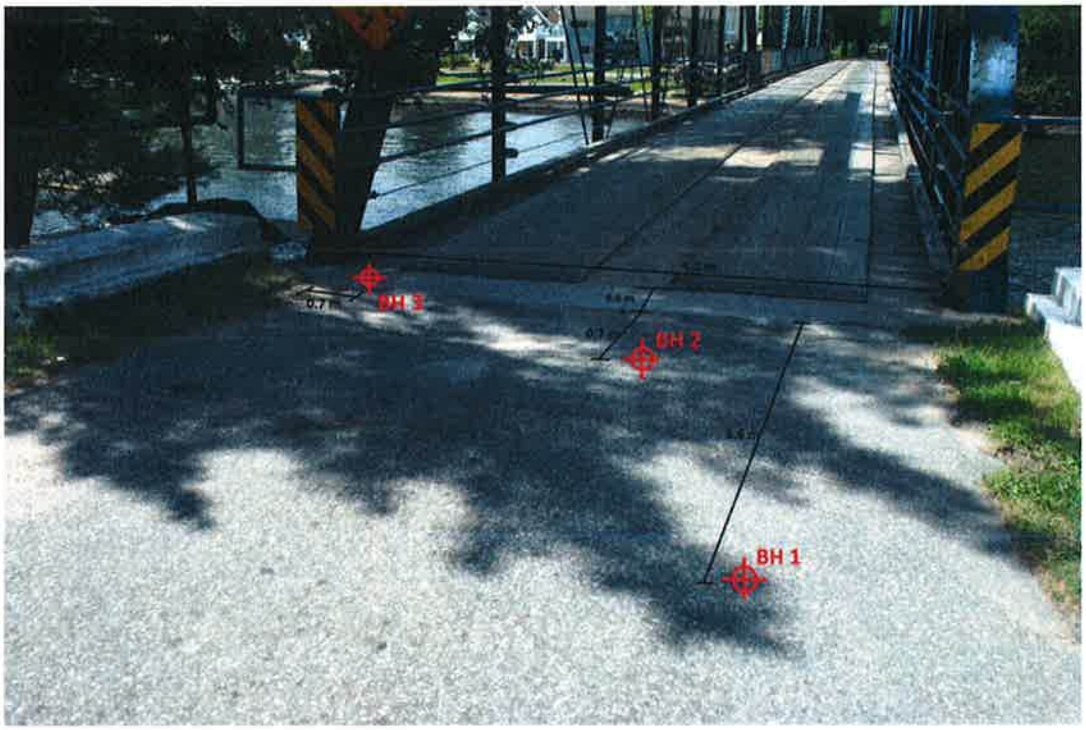
SITE PHOTOGRAPHS	Figure 13A

No.1: Overview of the Hamlet Bridge from north side of bridge, looking south; the east side (left on photo) is the fixed bridge; the west side (right on photo) is swing bridge.

No.2: Overview of the Hamlet Bridge, looking west; the closest pier is the east pier which supports the west end of fixed bridge and east end of the swing bridge.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13B



No.3: Overview of borehole locations and approximate measurements at east abutment, looking west.

No.4: Test pit location at south side of south wing wall at east abutment and approximate measurements, looking north.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13C

No.5: North wing wall of east abutment, looking south; the linear cracks appear to be construction joints.

No.6: The north abutment concrete wall under the bridge; note the sheet pile wall installed for erosion protection; note the concrete platform behind the sheet pile walls.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13D

No.7: The north abutment concrete wall under the bridge; the linear crack appears to be the construction joint.

No.8: The concrete platform between the sheet piles and abutment wall; the core hole was terminated in the concrete due to refusal on steel rebar at a depth of about 0.5 m.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13E

No.9: Rock cores in Borehole 2

No.10: Concrete cores in vertical corehole at Borehole 3 location.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13F

No.11 Concrete cores in vertical corehole at Borehole 3 location.

No.12: Concrete cores in vertical corehole at Borehole 3 location.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS	Figure 13G

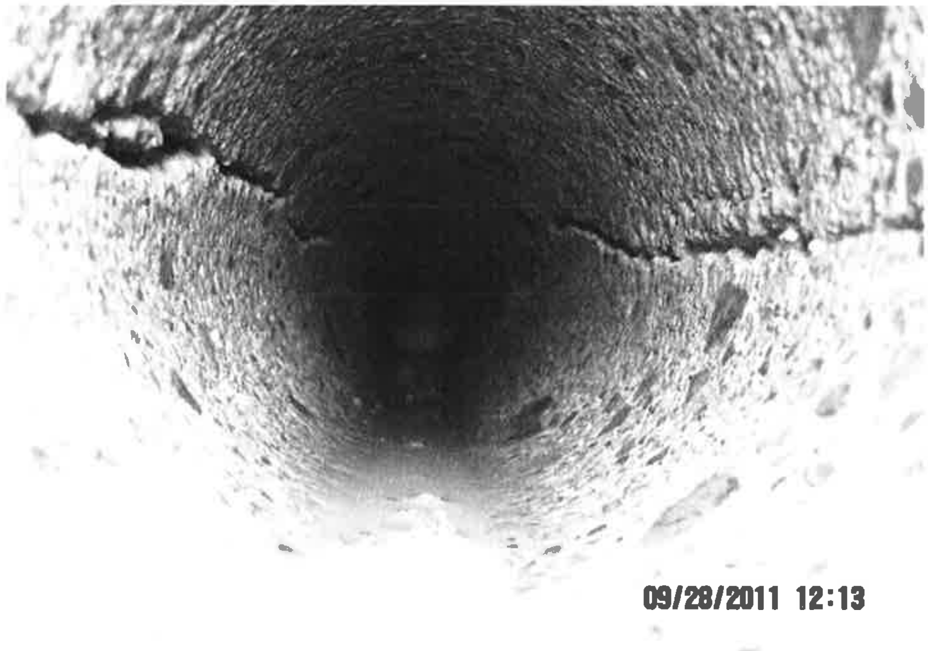
No.13: The cores of the horizontal corehole on the abutment wall.

No.14: The cores of the horizontal corehole on the abutment wall.

Project No.	11-1111-0118	Golder Associates	Inputted by:	az
Date:	March 2012		Checked by:	dbl

SITE PHOTOGRAPHS

Figure 13H



No.15: Looking into the horizontal concrete corehole on the abutment wall.

Project No.	11-1111-0118	**Golder Associates**	Inputted by:	az
Date:	March 2012		Checked by:	dbl

APPENDIX A

Important Information and Limitation of this Report

IMPORTANT INFORMATION AND LIMITATIONS TO THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder can not be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then upon the reasonable request of the client, Golder may authorize in writing the use of this report by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report. The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Sample Disposal: Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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solutions@golder.com
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100, Scotia Court
Whitby, Ontario, L1N 8Y6
Canada
T: +1 (905) 723 2727

Golder Associates

APPENDIX H

DETAILED CONDITION SURVEY

March 9, 2012

DETAILED BRIDGE CONDITION SURVEY

Hamlet Bridge, Trent Severn Waterway, Hamlet, Ontario

Submitted to:
Delcan Corporation
1223 Michael Street, Suite 100
Ottawa. Ontario
K1J 7T2

Attention: Mr. Patrick Mergel, P.Eng.

Report Number: 11-1111-0118

Distribution:

4 Copies - Delcan Corporation
1 e-Copy - Delcan Corporation
2 Copies - Golder Associates Ltd.

REPORT

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Table of Contents

ATTACHMENTS

Attachment 1
DETAILED CONDITION SURVEY SUMMARY SHEETS

SURVEY EQUIPMENT AND CALIBRATION PROCEDURES

FIELD AND LABORATORY TEST PROCEDURES

Attachment 2
CORE PHOTOGRAPHS AND SKETCHES

Attachment 3
CORE LOGS FOR SUBSTRUCTURE ELEMENTS

Attachment 4
SITE PHOTOS

Attachment 5
DRAWINGS

STRUCTURE IDENTIFICATION SHEET

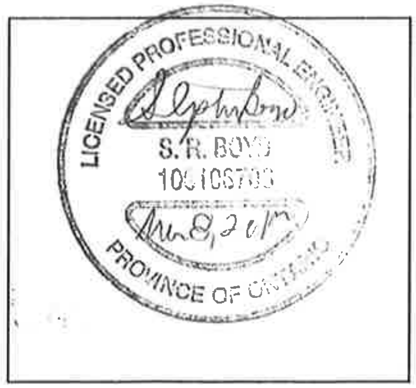
GENERAL INFORMATION

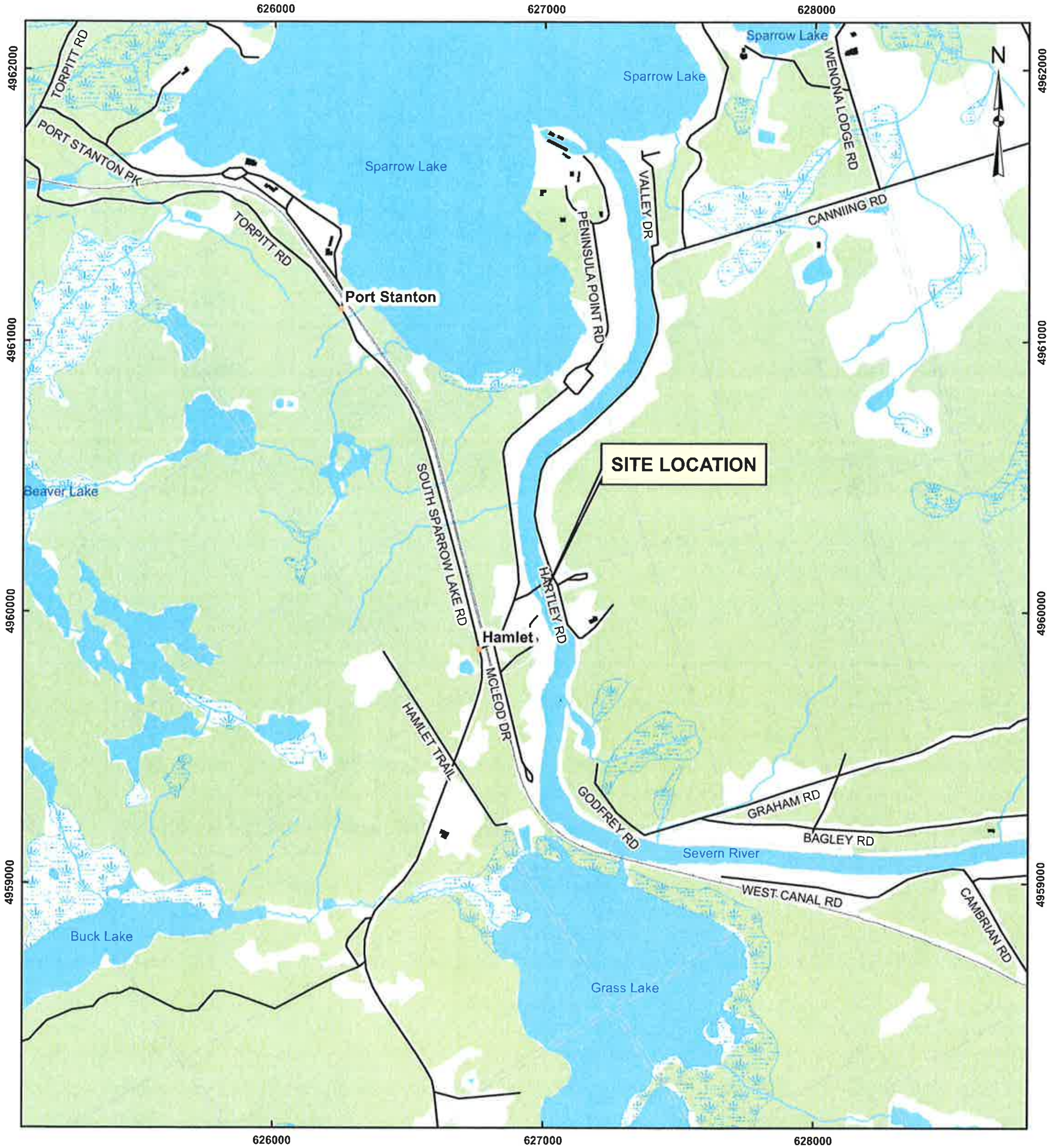
STRUCTURE NAME: Hamlet Bridge
MTO SITE NUMBER: N/A | DISTRICT NUMBER: N/A
HIGHWAY: Canning Road | below: Trent Severn Waterway
TYPE OF STRUCTURE: Steel through truss with a wooden deck
NUMBER OF SPANS: 2 | SPAN LENGTH (m): 61, 31
ROADWAY WIDTH (m): N/A | YEAR BUILT: originally 1920 and 1922

DIRECTION OF STRUCTURE: East-West
SEQUENCE NUMBER: N/A | TOWNSHIP NUMBER: N/A
LHRS NUMBER: N/A | BRIDGE NUMBER (MUNIC.): N/A
LOCATION: Hamlet, Ontario | JURISDICTION:
(See Key Plan, Figure 1)

INSPECTOR'S NAME: S. Jagdat, P.Eng.
PARTY MEMBERS: P. Barnhill, Z. Lin
DATE(s) OF INSPECTION: September 28, 2011
TEMPERATURE: 20°C | WEATHER: Rain
MTO REGION: N/A | AADT: N/A
DECK RIDING SURFACE: Wood
YEAR LAST REHABILITATED: 1985-1990

ENGINEER'S STAMP





400 200 0 400 800

SCALE 1:20,000 METRES

PROJECT

DELCAN
HAMLET BRIDGE (FIXED SPAN)
HAMLET, ONTARIO

TITLE

KEY PLAN

REFERENCE

Base Data - MNR NRVIS, obtained 2004, CANMAP v2006.4
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Projection: Transverse Mercator Datum: NAD 83 Coordinate System: UTM Zone 17

Golder Associates
Whitby, Ontario

PROJECT NO. 11-1111-0118		
DESIGN	SS	OCT. 2011
GIS	SS/JT	OCT. 2011
CHECK		
REVIEW		

SCALE AS SHOWN | REV. 0.0

FIGURE 1

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Delcan Corporation (Delcan) to carry out a detailed condition survey of the Hamlet Bridge substructure on Canning Road (Regional Road 49) over the Trent Severn Waterway. The bridge is located approximately 7 kilometres North of Hwy 11 in Hamlet, Ontario.

The condition survey and laboratory testing of core samples were carried out in accordance with MTO's "Structure Rehabilitation Manual, Part 1 – Condition Surveys" (April, 2007). A visual inspection and coring of the bridge abutments, wingwalls and centre pivot pier were carried out on September 28, 2011. The laboratory testing of the extracted concrete cores was completed in October 2011. A summary of the field and laboratory test procedures performed by Golder is included in Attachment 1 for reference. The results of the condition survey have been reported, where applicable, on standard MTO forms.

2.0 DESCRIPTION OF THE STRUCTURE

The existing bridge consists of a swing bridge on the west side and a fixed bridge on the east side of the waterway. The bridge is a steel through truss on a wooden deck. The bridge has an overall length of approximately 303 feet (200 feet for the swing bridge and 103 feet for the fixed bridge). The bridge was built between 1920 and 1922. It is understood that the west abutment (i.e. the abutment for the swing bridge) was reconstructed between 1985 and 1990. The bridge has two (2) intermediate piers. The west pier supports the central pivot to the swing bridge. The east pier supports the swing bridge and the fixed bridge.

3.0 SUMMARY OF SIGNIFICANT FINDINGS

3.1 Core Sampling and Testing

3.1.1 East Abutment and Wingwall Cores

A total of six (6) cores were taken from the east abutment and wingwalls of the Hamlet Bridge to evaluate the condition of the concrete. Two cores (Cores C1 and C2) were extracted from the east abutment and two cores (Cores C3 to C6) were extracted from each of the southeast and northeast wingwalls. The concrete cores were taken at the locations shown on Figure 5-1 in Attachment 5. Photographs and sketches of the cores are included in Attachment 2. Observation made while coring and the results of laboratory testing are summarized on the 'Core Log for Exposed Concrete Components' sheets located in Attachment 3. The cores from the east abutment and wingwalls were generally in good to fair condition, however there was a marked absence of coarse aggregate particles in the concrete as seen in the core photos in Attachment 2. Most of the aggregate in the core samples form the east abutment and wingwalls was lass than approximately 10 mm.

The cores were brought to Golder's laboratory in Whitby for visual assessment and selected laboratory testing. Tests performed included compressive strength, chloride ion content and air void parameter measurements. The results of the laboratory testing carried out on the cores extracted from the east abutment and wingwalls are presented in Sections 3.1.1.1 to Section 3.1.1.3.

3.1.1.1 Compressive Strength

Compressive strength testing was carried out on three (3) cores (Cores C1, C3, and C6) from the east abutment and wingwalls. The compressive strengths for the cores extracted from the east abutment, southeast wingwall and northeast wall were 10.7 MPa, 17.4 MPa and 12.5 MPa, respectively, resulting in an average compressive strength of 13.5 MPa for the three samples tested. According to the MTO Structural Rehabilitation Manual (April 2007), strength values less than 20 MPa represent poor quality concrete.

3.1.1.2 Chloride Ion Content

Chloride ion content testing was carried out on one core sample from the southeast wingwall (Core C4) as shown on the 'Core Logs for Asphalt Covered Bridge Decks' in Attachment 3. A background chloride ion value of 0.004% was selected after reviewing the chloride data. The chloride ion content was then corrected for this background level. The chloride ion content did not exceed the commonly accepted threshold for corrosion to occur (0.025 percent by mass of concrete) in core C4 to at least a depth 230 mm.

3.1.1.3 Air Void Parameter Measurements

Air void parameter measurements were carried out on two core samples, one from the east abutment and one from the northeast wingwall (Cores C2 and C5). The average measured air content, specific surface and spacing factor were 16.1 percent, 10.18 mm^2/mm^3 and 0.201 mm, respectively, as shown in Table 1 below. The Structure Rehabilitation Manual specifies that properly air entrained concrete should have an air content greater than 3 percent, a specific surface greater than 24.0 mm^2/mm^3 and a spacing factor less than 0.200 mm. Based on the results from the air void parameter measurements the cores do not satisfy the requirements for properly air-entrained concrete, mainly due to the low specific surface. The high air contents (average of 16.1%) could be one cause for the relatively low compressive strengths obtained.

Table 1: Air Void Parameter Measurements

Cores	Average measured air content (%)	Specific Surface (mm^2/mm^3)	Spacing Factor (mm)
C2	14.3	9.71	0.209
C5	17.9	10.64	0.193
Average	16.1	10.18	0.201

3.1.2 Northwest Wingwall Cores

A total of two (2) cores (Cores C7 and C8) were taken from the northwest wingwall of the Hamlet Bridge to evaluate the condition of the concrete. The concrete cores were taken at the locations shown on Figure 5-2 in Attachment 5. Photographs and sketches of the cores are included in Attachment 2 and the results of the sampling and laboratory testing are summarized on the 'Core Log for Exposed Concrete Components' sheets located in Attachment 3.

The cores from the northwest wingwall were generally in good to fair condition. The concrete was similar to the concrete cores from the east abutment and wingwalls but more coarse aggregate particles were observed in the cores from the northwest wingwall. One core, (Core C8) had a large cobble sized rock the full diameter of the core and extending from a depth of about 120 mm to the end of the core at 240 mm.

The cores were brought to Golder's laboratory in Whitby for visual assessment and selected laboratory testing. Tests performed included compressive strength, and chloride ion content measurements. The results of the laboratory testing carried out on the cores extracted from the northwest wingwall are presented in Section 3.1.2.1 and Section 3.1.2.2.

3.1.2.1 Compressive Strength

Compressive strength testing was carried out on one core (Core C7) from the northwest wingwall. The compressive strength for the core extracted from the northwest wingwall was 10.8 MPa. This is very similar to the compressive strengths of the cores tested from the east abutments and wingwalls.

3.1.2.2 Chloride Ion Content

Chloride ion content testing was carried out on one core sample from the northwest wingwall (Core C8) as shown on the 'Core Logs for Asphalt Covered Bridge Decks' in Attachment 3. A background chloride ion value of 0.007% was selected after reviewing the chloride data. The chloride ion content was then corrected for this background level. The chloride ion content did not exceed the commonly accepted threshold for corrosion to occur (0.025 percent by mass of concrete) at least to the depth tested (90 mm).

3.1.3 West Abutment Cores

A total of two (2) cores were taken from the Hamlet Bridge west abutment to evaluate the condition of the concrete. The two cores (Cores C9 and C10) were extracted from the west abutment at the locations shown on Figure 5-2 in Attachment 5. Photographs and sketches of the cores are included in Attachment 2. The results of the sampling and laboratory testing are summarized on the 'Core Log for Exposed Concrete Components' sheets located in Attachment 3.

The concrete cores from the west abutment were in good condition with no defects observed. The west abutment and the southwest wingwall had been rehabilitated and the concrete used in the west abutment (and the southwest wingwall) is different than the concrete in the east abutments and wingwalls and the northwest wingwall. The coarse aggregates are well proportioned and uniformly dispersed. Reinforcing steel was encountered in both cores from the west abutment at a depth of 130 mm in both cores C9 and C10.

The cores were brought to Golder's laboratory in Whitby for visual assessment and selected laboratory testing. Tests performed included compressive strength, and chloride ion content measurements. The results of the laboratory testing carried out on the cores extracted from the west abutment are presented in Section 3.1.3.1 and Section 3.1.3.2.

3.1.3.1 Compressive Strength

Compressive strength testing was carried out on two (2) cores (Core C9 and C10) from the west abutment. Core C10 was long enough that both a compressive strength sample and a chloride ion content sample could be obtained for the core. The compressive strength sample was taken from the interior portion of the core, from a depth of about 145 mm to 270 mm. The compressive strength for the cores extracted from the west abutment were 44.1 MPa and 38.3 MPa for cores C9 and C10, respectively, resulting in an average compressive strength of 41.2 MPa.

3.1.3.2 Chloride Ion Content

Chloride ion content testing was carried out on one core sample from the west abutment (Core C10) as shown on the 'Core Logs for Asphalt Covered Bridge Decks' in Attachment 3. According to the MTO Structure Rehabilitation Manual, April 2007, section 5.4.3 for determining the background chloride ion level "the lowest value should be similar in two successive slices of a core, and the background value should not exceed 0.07% by mass of concrete." A background chloride ion value of 0.059% was selected after reviewing the chloride data for Core C10. The chloride ion content was then corrected for this background level. The corrected chloride ion content in Core C10 did not exceed the commonly accepted threshold for corrosion to occur (0.025 percent by mass of concrete) below a depth of 30 mm.

3.1.4 Central Pivot Pier Cores

A total of four (4) cores were taken from the central pivot pier of the Hamlet Bridge to evaluate the condition of the concrete. Two cores (Cores C11 and C12) were extracted from the north side of the central pivot pier and two cores (Cores C13 to C14) were extracted from the south side of the pivot pier. The concrete cores were taken at the locations shown on Figure 5-3 in Attachment 5. Photographs and sketches of the cores are included in Attachment 2. The results of the sampling and laboratory testing are summarized on the 'Core Log for Exposed Concrete Components' sheets located in Attachment 3.

The cores from the central pivot pier were generally in good to fair condition. The aggregate in the concrete was well dispersed with some coarse aggregate particles having a maximum top size greater than 40 mm. Cracking in some of the larger coarse aggregate particles was observed, as shown on the inset on Figure 2-12 in Attachment 2. This cracking was most likely caused by freezing and thawing but there is a slight chance it was caused by alkali-silica reaction (ASR). ASR is a reaction between the alkalis in the cement and certain minerals in some aggregates. Further testing would be required to determine if the cracking was caused by ASR.

The cores were brought to Golder's laboratory in Whitby for visual assessment and selected laboratory testing. Tests performed included compressive strength, and chloride ion content measurements. The results of the laboratory testing carried out on the cores extracted from the central pivot pier are presented in Section 3.1.4.1 and Section 3.1.4.2.

3.1.4.1 Compressive Strength

Compressive strength testing was carried out on three (3) cores (Cores C11, C13, and C14) from the central pivot pier. The compressive strengths for the cores extracted from the central pivot pier were 23.5 MPa, 30.3 MPa, and 34.6 MPa, respectively for Cores C11, C12 and C14, resulting in an average compressive strength of 29.5 MPa for the three samples tested.

3.1.4.2 Chloride Ion Content

Chloride ion content testing was carried out on one core sample from the central pivot pier (Core C12) as shown on the 'Core Logs for Asphalt Covered Bridge Decks' in Attachment 3. A background chloride ion value of 0.009% was selected after reviewing the chloride data. The chloride ion content was then corrected for this background level. The corrected chloride ion content did not exceed the commonly accepted threshold for corrosion to occur (0.025 percent by mass of concrete) in core C12.

3.1.5 East Pier Cores

A total of two cores (Cores C15A and C15B) were extracted from the south side of the east pier for compressive strength testing. The cores from the east pier were generally in good to fair condition. The aggregate in the concrete was well dispersed with some coarse aggregate particles having a maximum top size greater than 40 mm. The compressive strength of Core 15A was 17.0 MPa and the compressive of Core 15B was 23.9 MPa.

3.2 Bridge Components

A member of Golder's engineering staff carried out a visual assessment of the exposed concrete components, and other elements of the structure. The results of the visual condition survey are summarized in sections 3.2.1 and 3.2.2.

3.2.1 Abutments and Wingwalls

Golder carried out a visual assessment of the abutments and wingwalls above the water line for the Hamlet Bridge. The overall condition of the abutments and wingwalls with the observed deterioration are shown on Figures 5-1 and 5-2, in Attachment 5.

The east abutment and wingwalls appear to be the original concrete from around 1920. Photographs of the southeast wingwall, east abutment and the northeast wingwall are shown on Figures 4-4, 4-5 and 4-6, respectively. The southeast and northeast wingwalls appear to be built in several sections and some of the joints have started to deteriorate. The concrete is relatively sound with no delaminations noted on the two wingwalls. The southeast wingwall exhibited some light scaling along the exposed section just above the ground line. The northeast wingwall exhibited some heavier scaling/spalling on the lower corner where the wingwall met the abutment. A few random cracks were also noted on the two wingwalls. The east abutment was in fair condition with several delaminated areas across the face of the abutment. Some spalling or heavy scaling was also observed, particularly on the northeast corner. Scouring of the surface by ice and debris during spring runoff may be the cause of most of the spalling/scaling observed.

The west end of the northwest wingwall is also original concrete and is in good to fair condition. Some scaling was observed on the surface but no delaminations were noted. A photograph of the northwest wingwall is shown on Figure 4-7.

The faces of the west abutment and the southwest wingwall were replaced during the rehabilitation around 1990 and are in good condition. A few short random cracks were observed at each end of the abutment. No significant scouring or spalling was observed. The southwest wingwall was in good condition with no significant deterioration observed. Photographs of the west abutment and the southwest wingwall are shown on Figures 4-8 and 4-9, respectively.

3.2.2 Central Pivot Pier

A visual condition survey was carried out on the top surface of the centre pivot pier. Photographs of the north side of the pivot pier and the pier extending north from the pivot pier are shown on Figures 4-10 to 4-12. Photographs of the south side of the pivot pier and the pier extending south from the pivot pier are shown on Figures 4-13 to 4-15. The overall condition of the top surface of the pivot pier is shown in Figure 5-3 in Attachment 5. The top surface of the pivot pier exhibited some spalling or heavy scaling over several areas. Random stained cracks were also observed, concentrated mostly in the northeast and southeast corners of the pivot pier.

The top surface of the two piers extending out form the center pivot pier were generally in good to fair condition with some surface scaling and a few scattered random cracks. The blocks at the ends of these piers, designed to support the ends of the swing bridge when it is in the open position exhibited significant cracking, especially on the south sides. This cracking could be caused by freezing and thawing, exacerbated by wetting and drying of the south faces.

4.0 CLOSING

We trust that this report is sufficient for your current needs. If you have any questions, or require any additional information, please do not hesitate to contact our office.

Yours truly,

GOLDER ASSOCIATES LTD.

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PEB/SRB/MLJM:peb/leb

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ATTACHMENT 1

DETAILED CONDITION SURVEY SUMMARY SHEETS
SURVEY EQUIPMENT AND CALIBRATION PROCEDURES
FIELD AND LABORATORY TEST PROCEDURES

DETAILED CONDITION SURVEY SUMMARY SHEET
ASPHALT COVERED BRIDGE DECKS

 Hamlet Bridge

Component Type and Location: Central Pivot Pier

OSIM IDENTIFIER:

1. DIMENSIONS AND AREA

Width(s) :7.3 Length(s): 4.85 m Height(s): 3.4 m

Diameter: Total Area Surveyed: ~49.0 m^2

2. CRACKS (MEDIUM AND WIDE)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	**Clean**	2.2	4.9	0.0	7.1 m
	Stained	0.0	0.0	0.0	
Wide Cracks	**Clean**	0.9	0.0	0.0	0.9 m
	Stained	0.0	0.0	0.0	

3. ALKALI AGGREGATE REACTION

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. DELAMINATIONS AND SPALLS

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.0	13.0	0.2
Total Delaminations and Spalls		**Total Delaminations and Spalls in Areas < –0.35 V**	
13.0 m^2	20.0 %	- m^2	- %

5. SCALING

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m^2
0.0	0.0	0.0	%

DETAILED CONDITION SURVEY SUMMARY SHEET
ASPHALT COVERED BRIDGE DECKS

 Hamlet Bridge

Component Type and Location: Central Pivot Pier

6. ADJUSTED CHLORIDE CONTENT PROFILE

Corrosion Activity at Core Location (V)		C 12		
Chloride Content (%)*	**0 – 10 mm**	0.016		
	20 – 30 mm	0.007		
	40 – 50 mm	0.009		
	60 – 70 mm	0.007		
	80 – 90 mm	0.000		
	100 – 110 mm			

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides from all cores taken in each range of corrosion potential.

7. CONCRETE COMPRESSIVE STRENGTH

Compressive Strength: 29.5 MPa (3 core tested)

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Page 1 of 2 Checked By: ____

DETAILED CONDITION SURVEY SUMMARY SHEET
ASPHALT COVERED BRIDGE DECKS

 Hamlet Bridge

Component Type and Location: East Pier (Top surface of south end only)

1. DIMENSIONS AND AREA

Width(s) : Length(s): Height(s):

Diameter: Total Area Surveyed: For Laboratory Testing only

2. ALKALI AGGREGATE REACTION

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

3. DELAMINATIONS AND SPALLS

Defect Type	**Delaminations**	**Spalls**	**Patches**
Area (m^2)	0.0	0.0	0.0
Total Delaminations and Spalls		**Total Delaminations and Spalls in Areas < –0.35 V**	
0.0 m^2	0.0 %	- m^2	- %

4. CONCRETE COMPRESSIVE STRENGTH

Compressive Strength: Average: 20 MPa (average)

17.0 MPa (Core C15A) 23.9 MPa (Core C15B)

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Page 1 of 1 Checked By: ____

Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: East Abutment

OSIM Identifier:

1. Dimensions and Area

Width(s) : Length(s): 6.4 m Height(s): 1.2 m

Diameter: Total Area Surveyed: ~9.6 m^2

2. Cracks (Medium and Wide)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	3.0	0.0	0.0	3.0 m
	Stained	0.0	0.0	0.0	

3. Alkali Aggregate Reaction

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. Delaminations and Spalls

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.92	1.46	0.0
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
2.38 m^2	25.0 %	- m^2	- %

5. Scaling

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m^2
0.0	0.0	0.0	%

Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: East Abutment

6. Concrete Air Entrainment

Concrete Air Entrained? Yes

Number of Cores Tested:	1
Average Air Content:	14.3 percent
Average Specific Surface:	9.71 mm^2/mm^3
Average Spacing Factor:	0.209 mm

7. Concrete Compressive Strength

Compressive Strength: 10.7 MPa (1 core tested)

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Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: West Abutment

OSIM Identifier:

1. Dimensions and Area

Width(s) : Length(s): 6.3 m Height(s): 0.8 m

Diameter: Total Area Surveyed: ~ 5.0 m^2

2. Cracks (Medium and Wide)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	

3. Alkali Aggregate Reaction

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. Concrete Cover (Based on core samples)

Minimum	Maximum	Average	
130	130	130	mm

Note: Includes only covers to top upper layer of rebars.

0 to 20 mm	0.0	40 to 60 mm	0.0	m^2
	0.0		0.0	%
20 to 40 mm	0.0	Over 60 mm	5.0	m^2
	0.0		100.0	%

Note: Only two measurement for concrete cover done.

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge

Component Type and Location: West Abutment

5. DELAMINATIONS AND SPALLS

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.0	0.0	0.0
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
0.0 m^2	0.0 %	- m^2	- %

6. SCALING

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m^2
0.0	0.0	0.0	%

7. HONEYCOMBING

Total Area: 0.0 m^2

8. ADJUSTED CHLORIDE CONTENT PROFILE

Corrosion Activity at Core Location (V)		C 10		
Chloride Content (%)*	0 – 10 mm	0.026		
	20 – 30 mm	0.054		
	40 – 50 mm	0.007		
	60 – 70 mm	0.002		
	80 – 90 mm	0.000		
	100 – 110 mm			

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides from all cores taken in each range of corrosion potential.

9. ADJUSTED CHLORIDE CONTENT AT LEVEL OF REBAR *

Core №	C10					
Chloride Content (%)**	0.000					

* Depth to reinforcing steel from core samples was 130 mm.

** Average chloride content as percent chloride by weight of concrete after deducting background chlorides

Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: West Abutment

10. Concrete Air Entrainment

Concrete Air Entrained? Yes (based on visual examination of core samples)

Number of Cores Tested: 0
Average Air Content:
Average Specific Surface:
Average Spacing Factor:

11. Concrete Compressive Strength

Compressive Strength: 41.2 MPa (2 core tested)

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Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: Southeast Wingwall

OSIM Identifier:

1. Dimensions and Area

Width(s) : Length(s): 2.42 m Height(s): 1 m

Diameter: Total Area Surveyed: ~3.0 m^2

2. Cracks (medium and wide)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	

3. Alkali Aggregate Reaction

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. Delaminations and Spalls

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.0	0.05	0.0
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
0.05 m^2	1.6%	0.0 m^2	0.0 %

5. Scaling

Light	Medium	Severe to Very Severe	
0.0	3.0	0.0	m^2
0.0	20.0	0.0	%

6. Honeycombing

Total Area: 0.0 m^2

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge

Component Type and Location: Southeast Wingwall

7. ADJUSTED CHLORIDE CONTENT PROFILE

		C 4		
Chloride Content (%)*	0 – 10 mm	0.002		
	20 – 30 mm	0.004		
	40 – 50 mm	0.011		
	60 – 70 mm	0.003		
	80 – 90 mm	0.001		
	100 – 110 mm			
	190 – 200 mm	0.003		
	220 – 230 mm	0.000		

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides from all cores taken in each range of corrosion potential.

8. ADJUSTED CHLORIDE CONTENT AT LEVEL OF REBAR

Core №	C4			
Chloride Content (%)*	-			

No rebar encountered within testing depth.

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides

9. CONCRETE AIR ENTRAINMENT

Concrete Air Entrained? Yes (based on the lab results of the core from the east abutment)

Number of Cores Tested:
Average Air Content:
Average Specific Surface:
Average Spacing Factor:

10. CONCRETE COMPRESSIVE STRENGTH

Compressive Strength:17.4 MPa (1 core tested)

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Page 1 to 2 Checked By: ____

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge

Component Type and Location: Southwest Wingwall

OSIM IDENTIFIER:

1. DIMENSIONS AND AREA

Width(s) : Length(s): 2.5 m Height(s): 1.4 m

Diameter: Total Area Surveyed: ~ 3.22 m²

2. CRACKS (MEDIUM AND WIDE)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	

3. ALKALI AGGREGATE REACTION

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m²

4. DELAMINATIONS AND SPALLS

Defect Type	Delaminations	Spalls	Patches
Area (m²)	0.0	0.0	0.0
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
0.0 m²	0.0 %	- m²	- %

5. SCALING

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m²
0.0	0.0	0.0	%

6. HONEYCOMBING

Total Area: 0.0 m²

Detailed Condition Survey Summary Sheet
Exposed Concrete Components

 Hamlet Bridge

Component Type and Location: Southwest Wingwall

7. Concrete Air Entrainment

Concrete Air Entrained? Yes (based on examination of cores samples form west abutment)

Number of Cores Tested: 0
Average Air Content:
Average Specific Surface:
Average Spacing Factor:

8. Concrete Compressive Strength

Average Compressive Strength: (no core tested)

Inputted By: PEB
Page 1 to 2 Checked By: ____

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

Hamlet Bridge

Component Type and Location: Northeast Wingwall

OSIM IDENTIFIER:

1. DIMENSIONS AND AREA

Width(s) :

Length(s): 2.45 m

Height(s): 1.5 m

Diameter:

Total Area Surveyed: ~ 4.4 m^2

2. CRACKS (MEDIUM AND WIDE)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	

3. ALKALI AGGREGATE REACTION

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. DELAMINATIONS AND SPALLS

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.0	0.5	0.0
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
0.5 m^2	11.3%	0.0 m^2	0.0 %

5. SCALING

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m^2
0.0	0.0	0.0	%

6.

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

Hamlet Bridge

Component Type and Location: Northeast Wingwall

7. CONCRETE AIR ENTRAINMENT

Concrete Air Entrained? Yes

Number of Cores Tested:	1
Average Air Content:	17.9 percent
Average Specific Surface:	10.64 mm^2/mm^3
Average Spacing Factor:	0.193 mm

8. CONCRETE COMPRESSIVE STRENGTH

Average Compressive Strength: 12.5 MPa (1 core tested)

Inputted By: PEB
Page 1 to 2 Checked By: ____

Detailed Condition Survey Summary Sheet
Exposed Concrete Components

Hamlet Bridge

Component Type and Location: Northwest Wingwall

OSIM Identifier:

1. Dimensions and Area

Width(s) : Length(s): 2.5 m Height(s): 0.9 m

Diameter: Total Area Surveyed: ~ 2.1 m^2

2. Cracks (Medium and Wide)

Type		Transverse	Longitudinal	Other - Map	Total
Medium Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	
Wide Cracks	Clean	0.0	0.0	0.0	0.0 m
	Stained	0.0	0.0	0.0	

3. Alkali Aggregate Reaction

Area of component with suspected severe to very severe alkali aggregate reaction: 0.0 m^2

4. Delaminations and Spalls

Defect Type	Delaminations	Spalls	Patches
Area (m^2)	0.0	0.03	0.02
Total Delaminations and Spalls		Total Delaminations and Spalls in Areas < –0.35 V	
0.03 m^2	1.4 %	0.0 m^2	0.0 %

5. Scaling

Light	Medium	Severe to Very Severe	
0.0	0.0	0.0	m^2
0.0	0.0	0.0	%

6. Honeycombing

Total Area: 0.0 m^2

DETAILED CONDITION SURVEY SUMMARY SHEET
EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge

Component Type and Location: Northwest Wingwall

7. ADJUSTED CHLORIDE CONTENT PROFILE

Corrosion Activity at Core Location (V)		0 to –0.19	–0.20 to –0.35	< –0.35
Chloride Content (%)*	0 – 10 mm	0.000		
	20 – 30 mm	0.008		
	40 – 50 mm	0.002		
	60 – 70 mm	0.007		
	80 – 90 mm	0.008		
	100 – 110 mm			

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides from all cores taken in each range of corrosion potential.

8. ADJUSTED CHLORIDE CONTENT AT LEVEL OF REBAR

Core №			No rebar encountered.	
Chloride Content (%)*				

* Average chloride content as percent chloride by weight of concrete after deducting background chlorides

9. CONCRETE AIR ENTRAINMENT

Concrete Air Entrained? Yes (based on the lab results of the core from the east abutment)

Number of Cores Tested: 0
Average Air Content:
Average Specific Surface:
Average Spacing Factor:

10. CONCRETE COMPRESSIVE STRENGTH

Average Compressive Strength: 10.8 MPa (1 core tested)

Inputted By: PEB
Page 1 to 2 Checked By: ____

FIELD AND LABORATORY TEST PROCEDURES

A – Core Sampling

Core samples with a nominal diameter of 94 mm are extracted from the deck to evaluate the condition of the concrete. All the cores are taken to our Whitby materials laboratory for visual assessment and laboratory testing of selected cores. Typical laboratory testing includes compressive strength and chloride ion content and air void system analysis. These test procedures are described in the paragraphs below.

All core holes are repaired at the end of each working day. The core holes are filled with a rapid setting concrete grout (Set 45) to the level of the original concrete surface. Where the core samples are extracted through an asphalt wearing surface, a Bituthene 3000 membrane is placed and the edges of the membrane sealed with Bituthene mastic. The asphalt course is then restored with commercial cold patch asphalt and compacted by a mechanical compactor.

Compressive Strength Testing

Compressive strength testing is carried out on selected core samples. The cores are trimmed and soaked for 48 hours prior to testing. The results indicated on the core logs have been corrected for the length to diameter ratio in accordance with CAN/CSA A23.2–00–14C, Table 1.

Chloride Ion Content Testing

Chloride ion content testing is carried out on selected core samples in accordance with Ministry of Transportation procedure LS-417. The *Structure Rehabilitation Manual* states that the chloride ion content threshold value necessary to depassivate steel and permit corrosion (in the presence of oxygen and moisture) is usually taken as 0.20 percent by mass of cement, or about 0.025 percent by mass of concrete (for typical concrete). The laboratory test procedure measures total chloride ion content. However, the total chloride ion content must be corrected for 'background' chloride ion levels found in the original concrete mix. Typically, the background chloride ion content for concrete from southern Ontario does not exceed 0.07 percent by mass of concrete.

Air Void System Analysis

Air void system testing is carried out on selected cores in accordance with ASTM procedure C457. The *Structure Rehabilitation Manual* states that properly air entrained concrete will exhibit the following properties: air content in excess of 3.0 percent; a spacing factor less than 0.200 mm; and a specific surface in excess of 24.00 mm^2/mm^3.

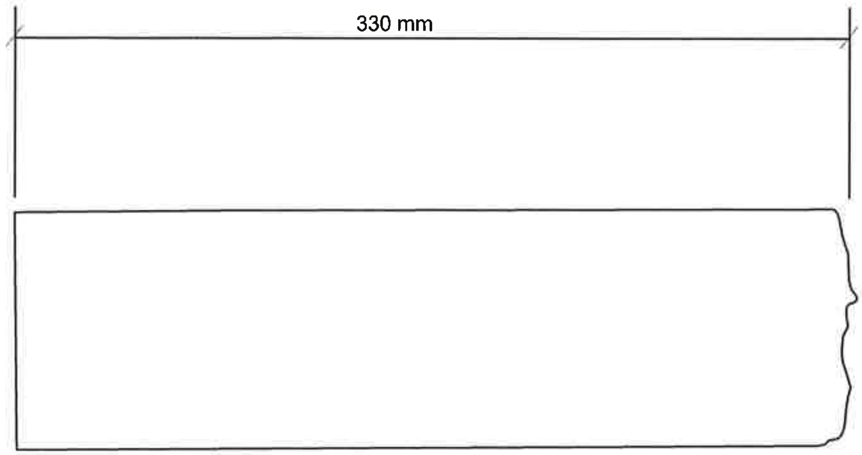
ATTACHMENT 2

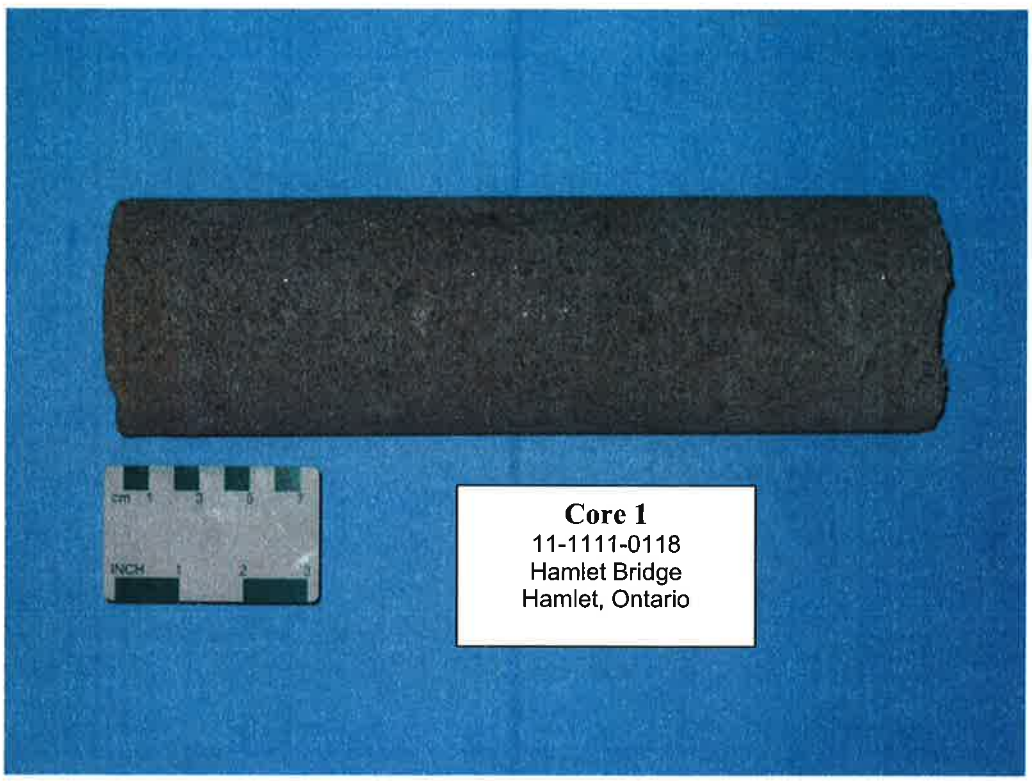
CORE PHOTOGRAPHS AND SKETCHES

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-1





Date:	March, 2012
Project:	11-1111-0118

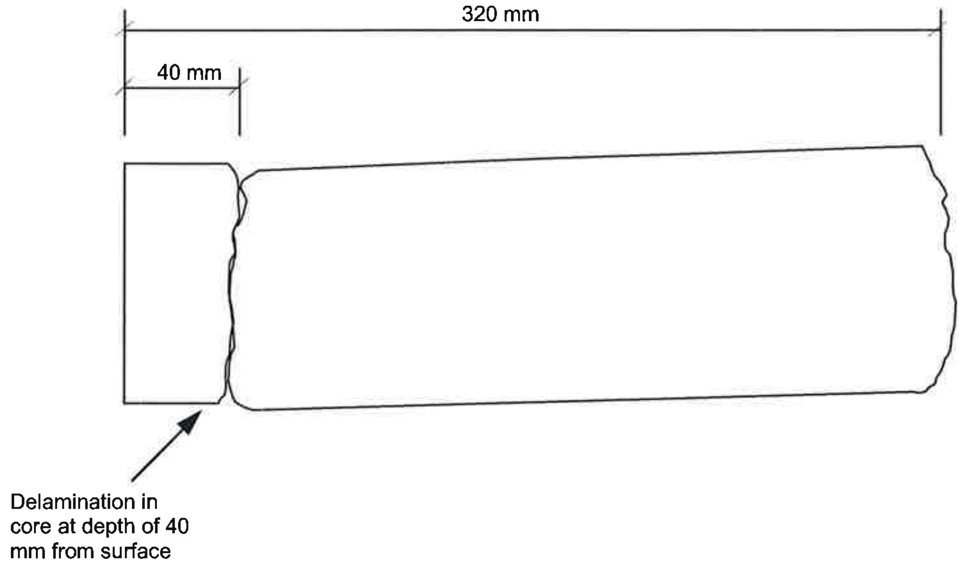
Golder Associates Ltd.

Inputted By:	PEB
Checked By:	

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-2



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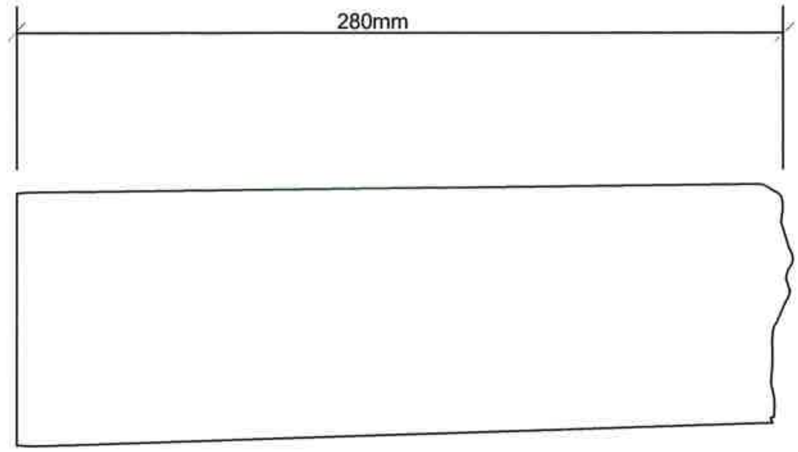
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-3

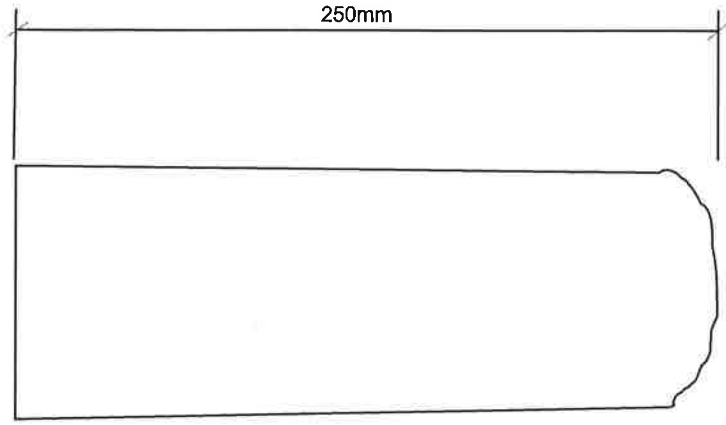


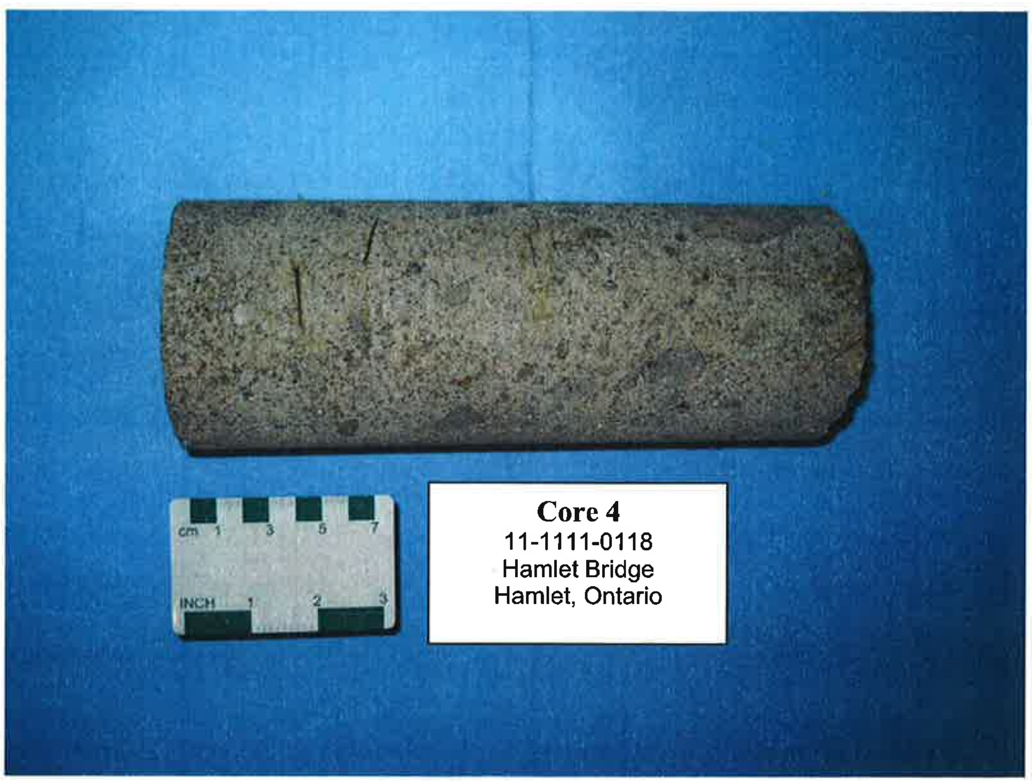
Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-4





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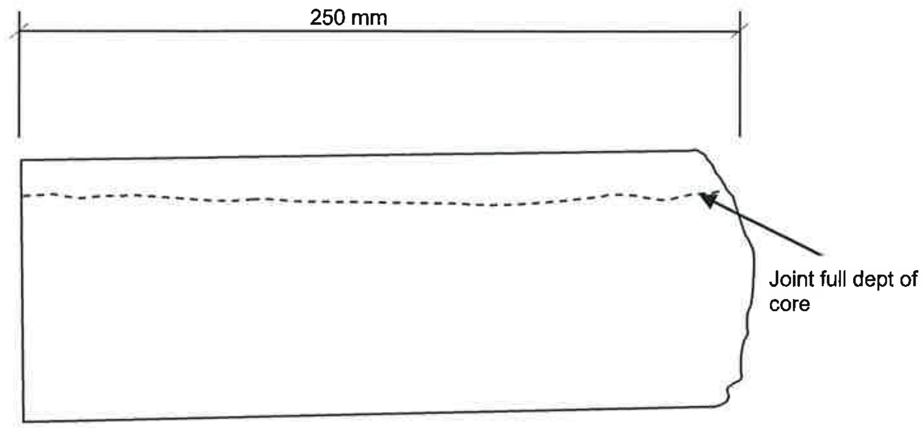
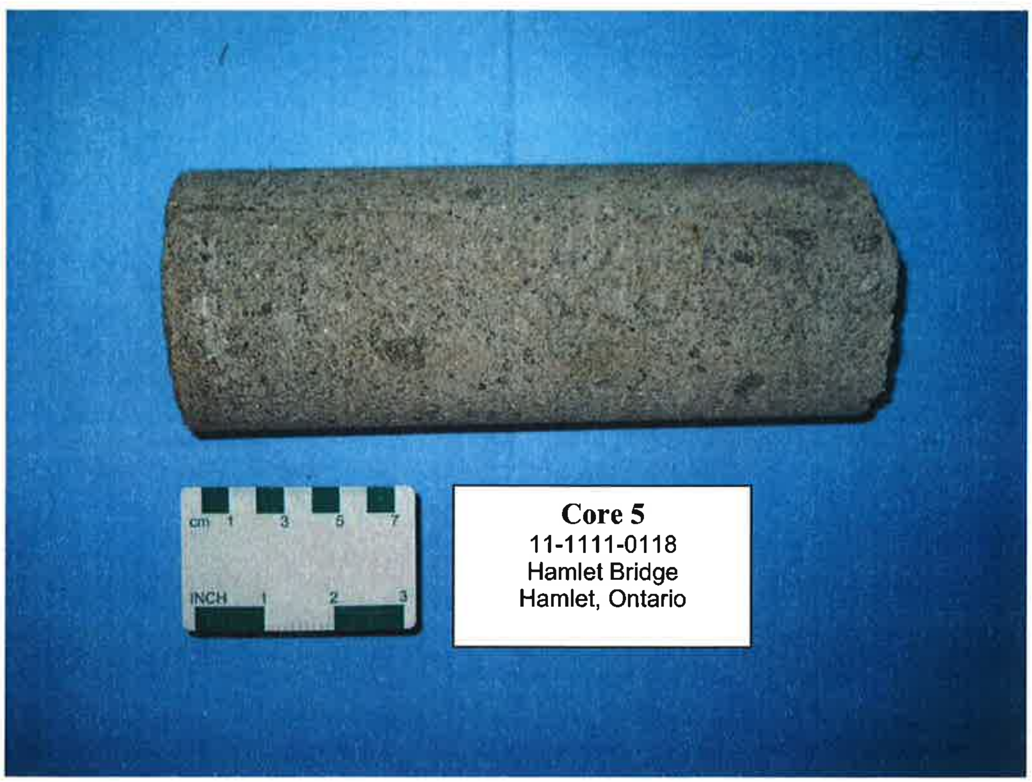
Checked By:	____

Core Photographs and Sketches Hamlet Bridge Hamlet, Ontario	Figure 2-5



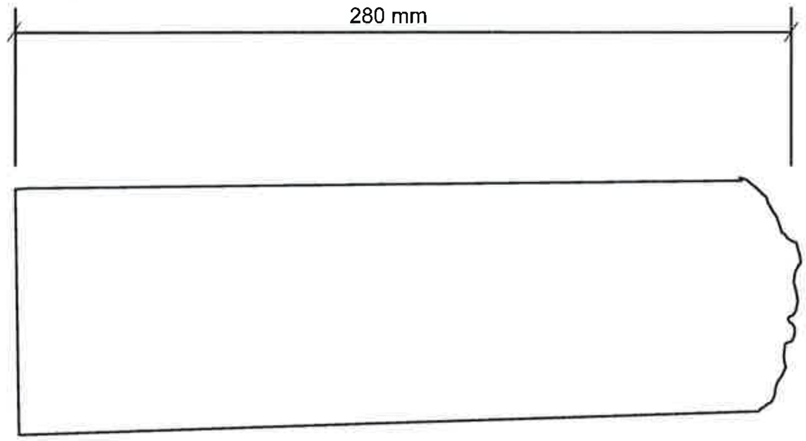


Date:	March, 2012	Golder Associates Ltd.	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-6

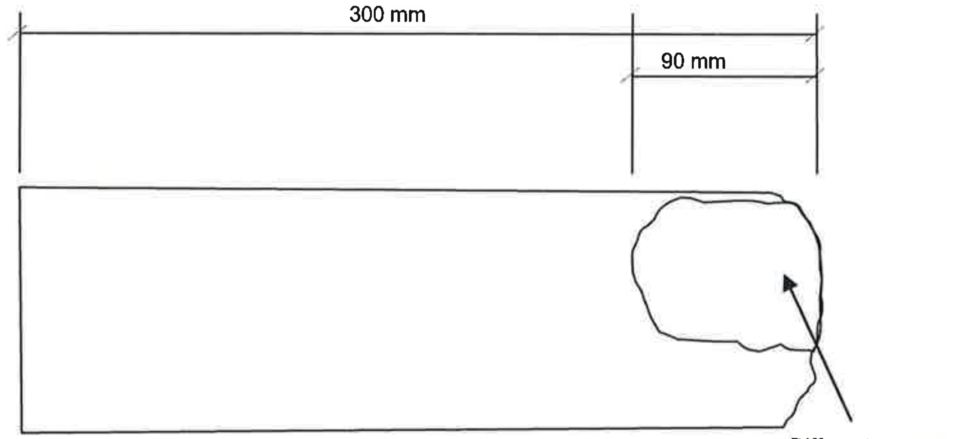


Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	___

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-7



Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
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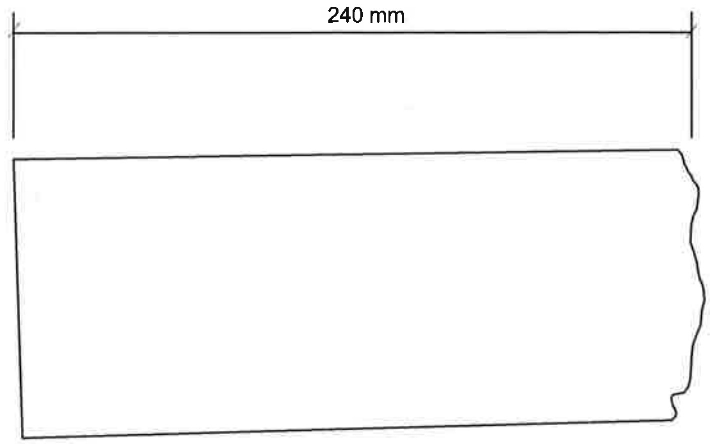
Core Photographs and Sketches

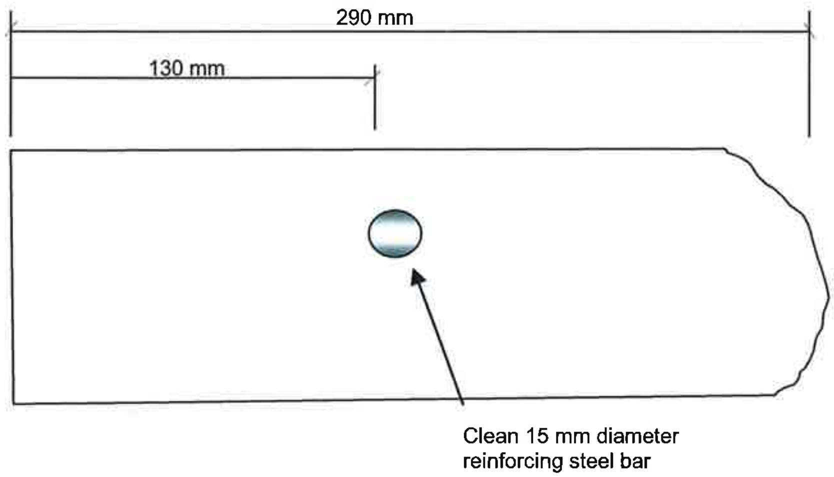
Hamlet Bridge
Hamlet, Ontario

Figure 2-8



Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches Hamlet Bridge Hamlet, Ontario	Figure 2-9

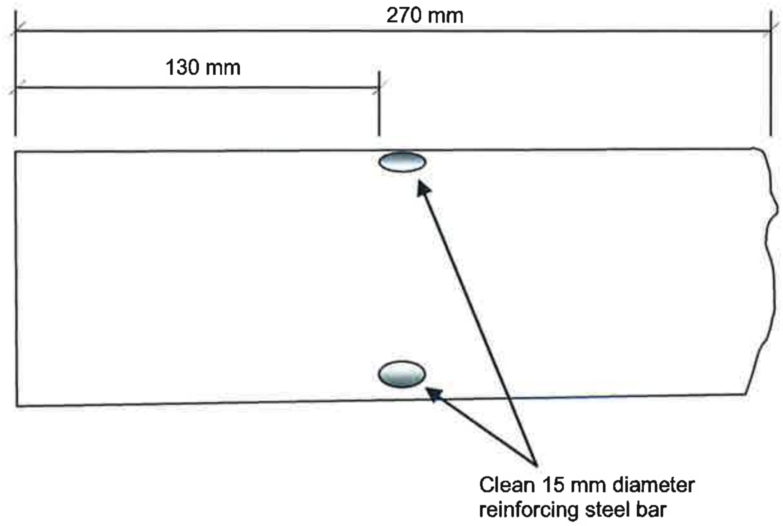


Date:	March, 2012	Golder Associates Ltd.	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-10

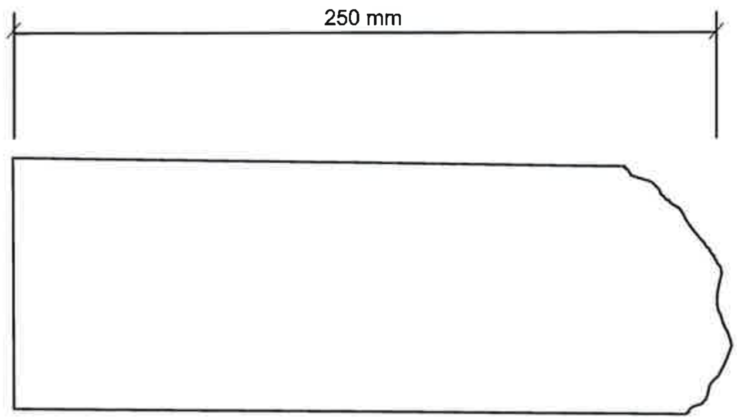


Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-11

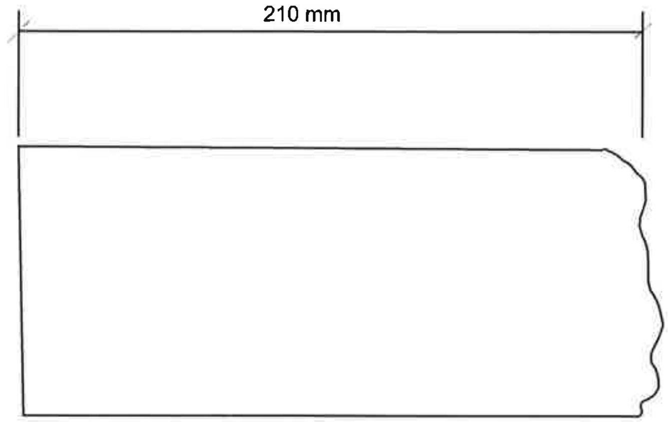


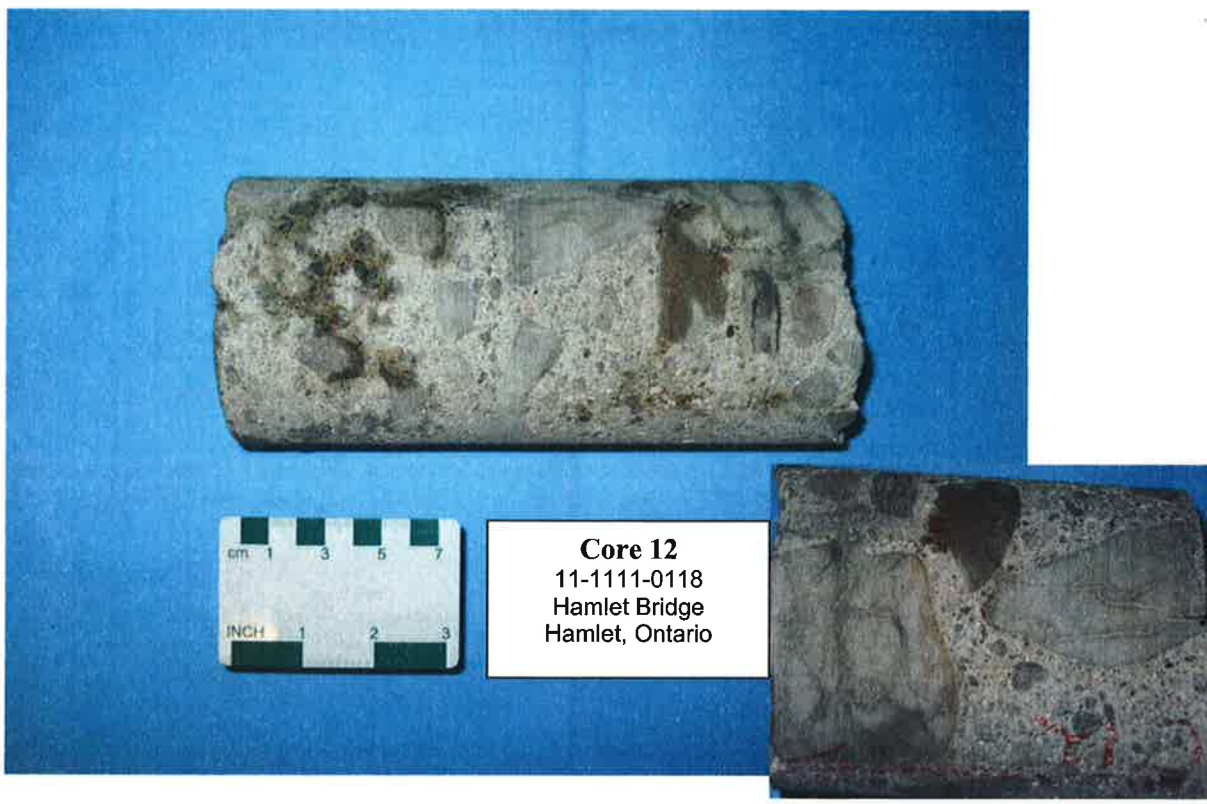
Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-12



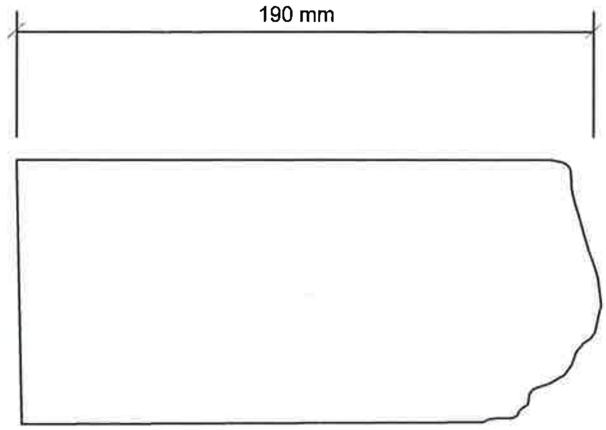


Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-13



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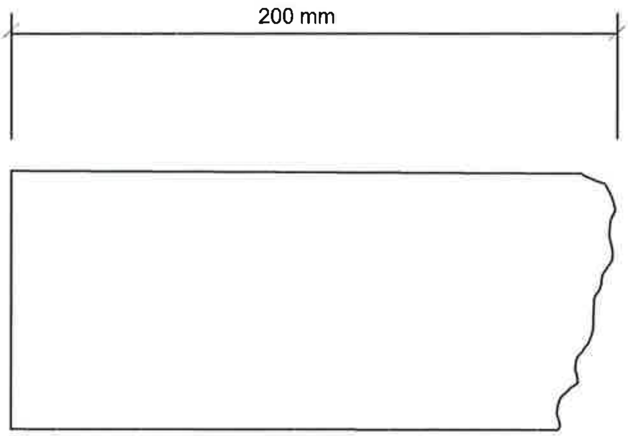
Project:	11-1111-0118		Checked By:	

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-14

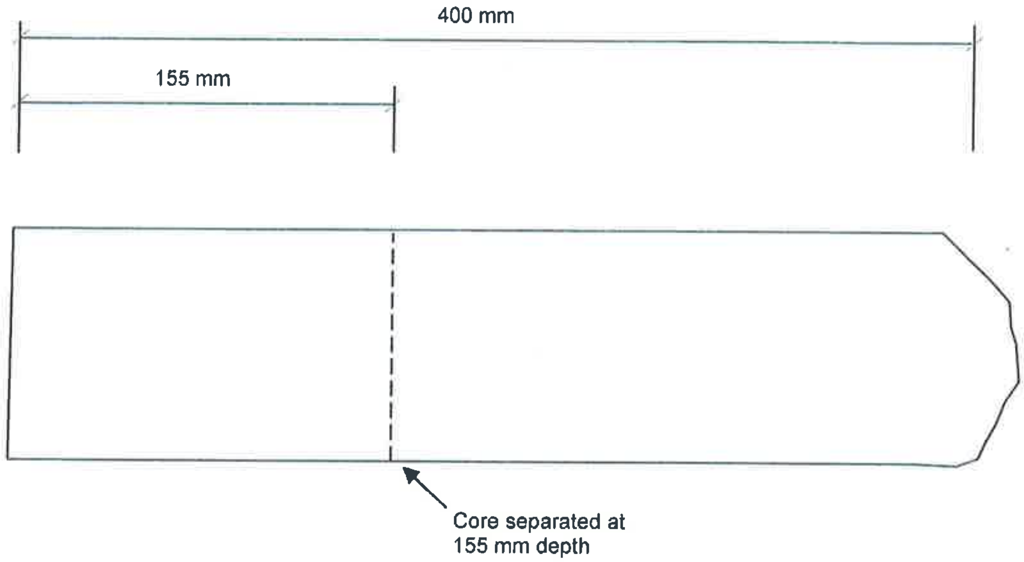


Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-15A



Date:	March, 2012
Project:	11-1111-0118

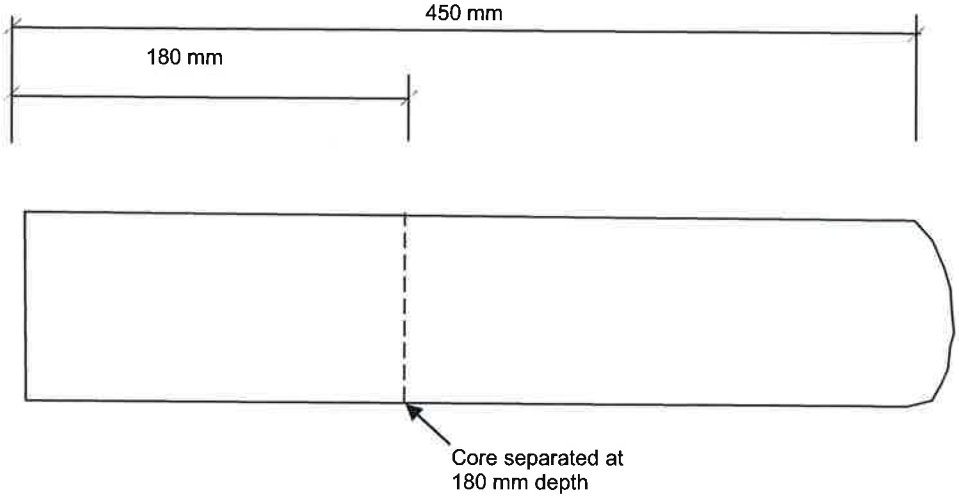
Golder Associates Ltd.

Inputted By:	PEB
Checked By:	____

Core Photographs and Sketches

Hamlet Bridge
Hamlet, Ontario

Figure 2-15B



Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

ATTACHMENT 3

CORE LOGS FOR SUBSTRUCTURE ELEMENTS

CORE LOG FOR EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge Bridge №

Core No.		C1	C2	C3
Location (Station, Gridline), m		East Abutment	East Abutment	Southeast Wingwall
Diameter, mm		94	94	94
Length, mm		330	320	280
Full Depth		N	N	N
Condition of Concrete [1]		G	F	G
Defects in Concrete		N	Y	N
Condition of Reinforcing steel bar [2]		N/A	N/A	N/A
Corrosion Potential at Nearest Grid Point, V		N/A	N/A	N/A
Compressive Strength, MPa		10.7		17.4
Chloride Content % Chloride by Weight of Concrete	Horizon 0–10 mm 20–30 mm 40–50 mm 60–70 mm 80–90 mm			
Air Voids	Air Content, %		14.3	
	Specific Surface, mm^2/mm^3		9.71	
	Spacing Factor, mm		0.209	
Testing Laboratory		Golder Associates Ltd.	Golder Associates Ltd.	Golder Associates Ltd.
Remarks		No defects noted in concrete core. Mostly fine aggregate in core, no coarse aggregate visible.	Concrete core delaminated at a depth of 40 mm. A few scattered coarse aggregate particles but mostly fine aggregate.	No defects noted in concrete core. 20 mm layer at surface of concrete with noted discoloration. A few scattered coarse aggregate particles but mostly fine aggregate.

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

CORE LOG FOR EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge Bridge №

Core No.		C4		C5	C6
Location (Station, Gridline), m		Southeast Wingwall		Northeast Wingwall	Northeast Wingwall
Diameter, mm		94		94	94
Length, mm		250		250	280
Full Depth		N		N	N
Condition of Concrete [1]		G		F	G
Defects in Concrete		N		N	N
Condition of Reinforcing steel bar[2]		N/A		N/A	N/A
Corrosion Potential at Nearest Grid Point, V		N/A		N/A	N/A
Compressive Strength, MPa					12.5
Chloride Content % Chloride by Weight of Concrete	Horizon 0–10 mm 20–30 mm 40–50 mm 60–70 mm 80–90 mm 100-110 mm 190-200 mm 220-230 mm	Measured 0.006 0.008 0.015 0.007 0.005 0.007 0.004	Corrected 0.002 0.004 0.011 0.003 0.001 0.003 0.000		
Air Voids	Air Content, %			17.9	
	Specific Surface, mm^2/mm^3			10.64	
	Spacing Factor, mm			0.193	
Testing Laboratory		Golder Associates Ltd.		Golder Associates Ltd.	Golder Associates Ltd.
Remarks		No defects noted in concrete surface. 30 mm layer at surface of concrete with noted discoloration. Some coarse aggregate particles but mostly fine aggregate.		Joint extending to full depth of core. A few scattered coarse aggregate particles but mostly fine aggregate.	No defects noted in concrete core. A few scattered coarse aggregate particles but mostly fine aggregate.

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

CORE LOG FOR EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge Bridge №

Core No.		C7	C8		C9
Location (Station, Gridline), m		Northwest Wingwall	Northwest Wingwall		West Abutment
Diameter, mm		94	94		94
Length, mm		300	240		290
Full Depth		N	N		N
Condition of Concrete [1]		F-G	G		G
Defects in Concrete		Y	N		N
Condition of Reinforcing steel bar[2]		N/A	N/A		C
Corrosion Potential at Nearest Grid Point, V		N/A	N/A		N/A
Compressive Strength, MPa		10.8			44.1
Chloride Content % Chloride by Weight of Concrete	Horizon		Measured	Corrected	
	0–10 mm		0.007	0.000	
	20–30 mm		0.015	0.008	
	40–50 mm		0.010	0.002	
	60–70 mm		0.014	0.007	
	80–90 mm		0.015	0.008	
	100-110 mm				
	105-115 mm				
	140-150 mm				
	160-170 mm				
Air Voids	Air Content, %				
	Specific Surface, mm^2/mm^3				
	Spacing Factor, mm				
Testing Laboratory		Golder Associates Ltd.	Golder Associates Ltd.		Golder Associates Ltd.
Remarks		Different concrete on corner of core from 210 – 300 mm depths. A few scattered coarse aggregate particles but mostly fine aggregate.	No defects noted in concrete core. Large cobble or boulder in core from 120 mm depth to end of core. Rest of coarse has few coarse aggregate particles.		No defects noted in concrete core. Clean 15 mm reinforcing steel bar at 130 mm cover. Well dispersed coarse aggregate. Max size approximately 20 mm.

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

CORE LOG FOR EXPOSED CONCRETE COMPONENTS

Core No.		C10		C11	C12	
Location (Station, Gridline), m		West Abutment		Central Pivot Pier	Central Pivot Pier	
Diameter, mm		94		94	94	
Length, mm		270		250	210	
Full Depth		N		N	N	
Condition of Concrete [1]		G		F-G	F-G	
Defects in Concrete		N		Y	Y	
Condition of Reinforcing steel bar[2]		C		N/A	N/A	
Corrosion Potential at Nearest Grid Point, V		N/A		N/A	N/A	
Compressive Strength, MPa		38.3		23.5		
Chloride Content % Chloride by Weight of Concrete	Horizon	Measured	Corrected		Measured	Corrected
	0–10 mm	0.085	0.026		0.025	0.016
	20–30 mm	0.113	0.054		0.016	0.007
	40–50 mm	0.066	0.007		0.018	0.009
	60–70 mm	0.061	0.002		0.016	0.007
	80–90 mm	0.059	0.000		0.009	0.000
	100-110 mm					
	105-115 mm					
	140-150 mm					
	160-170 mm					
Air Voids	Air Content, %					
	Specific Surface, mm^2/mm^3					
	Spacing Factor, mm					
Testing Laboratory		Golder Associates Ltd.		Golder Associates Ltd.	Golder Associates Ltd.	
Remarks		No defects noted in concrete core. Clean 15 mm reinforcing steel bar at 130 mm depth. Well dispersed coarse aggregate. Max size approximately 20 mm.		Surface of concrete moderately-lightly scaled or spalled. Cracks in aggregate possibly due to freezing and thawing. Some coarse aggregate particles, max size >40mm.	Surface of concrete moderately-lightly scaled or spalled. Cracks in aggregate possibly due to freezing and thawing. Some coarse aggregate particles, max size >40mm.	

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

CORE LOG FOR EXPOSED CONCRETE COMPONENTS

 Hamlet Bridge Bridge №

Core No.		C13	C14	
Location (Station, Gridline), m		Central Pivot Pier	Central Pivot Pier	
Diameter, mm		94	94	
Length, mm		190	200	
Full Depth		N	N	
Condition of Concrete [1]		G	G	
Defects in Concrete		N	N	
Condition of Reinforcing steel bar[2]		N/A	N/A	
Corrosion Potential at Nearest Grid Point, V		N/A	N/A	
Compressive Strength, MPa		30.3	34.6	
Chloride Content % Chloride by Weight of Concrete	Horizon 0–10 mm 20–30 mm 40–50 mm 60–70 mm 80–90 mm 100-110 mm 105-115 mm 140-150 mm 160-170 mm			
Air Voids	Air Content, %			
	Specific Surface, mm^2/mm^3			
	Spacing Factor, mm			
Testing Laboratory		Golder Associates Ltd.	Golder Associates Ltd.	
Remarks		Surface of concrete lightly scaled or spalled. No significant defects in core body. Some coarse aggregate particles, max size >40mm.	Surface of concrete lightly scaled or spalled. No significant defects in core body. Some coarse aggregate particles, max size >40mm.	

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

Core Log for Exposed Concrete Components

 Hamlet Bridge Bridge №

Core No.		C15A	C15B	
Location (Station, Gridline), m		East Pier	East Pier	
Diameter, mm		94	94	
Length, mm		155	450	
Full Depth		N	N	
Condition of Concrete [1]		F	F	
Defects in Concrete		N	N	
Condition of Reinforcing steel bar[2]		N/A	N/A	
Corrosion Potential at Nearest Grid Point, V		N/A	N/A	
Compressive Strength, MPa		17.0	23.9	
Chloride Content % Chloride by Weight of Concrete	Horizon 0–10 mm 20–30 mm 40–50 mm 60–70 mm 80–90 mm 100-110 mm 105-115 mm 140-150 mm 160-170 mm			
Air Voids	Air Content, %			
	Specific Surface, mm^2/mm^3			
	Spacing Factor, mm			
Testing Laboratory		Golder Associates Ltd.	Golder Associates Ltd.	
Remarks		Surface of concrete lightly scaled or spalled. Core separated at 155 mm depth. No significant defects in core body. Some coarse aggregate particles, max size >40mm.	Surface of concrete lightly scaled or spalled. Core separated at 180 mm depth. No significant defects in core body. Some coarse aggregate particles, max size >40mm.	

1 – G = Good, F = Fair, P = Poor
2 – Condition of Reinforcing steel bar: C = Clean, LR = Light Rust, MR = Moderate Rust, SR = Severe Rust, N/A = No Reinforcing steel bar Exposed
Condition of Epoxy Coating: ECG = Good, ECF = Fair, ECP = Poor – rusted and debonded areas

Inputted By: PEB
Pages 1 to 6 Checked By: ____

ATTACHMENT 4

SITE PHOTOS

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-1 Overview of bridge deck facing west.

Figure 4-2 Overview of fixed bridge section facing west.

Date: March, 2012	Golder Associates Ltd.	Inputted By: PEB
Project: 11-1111-0118		Checked By: ____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

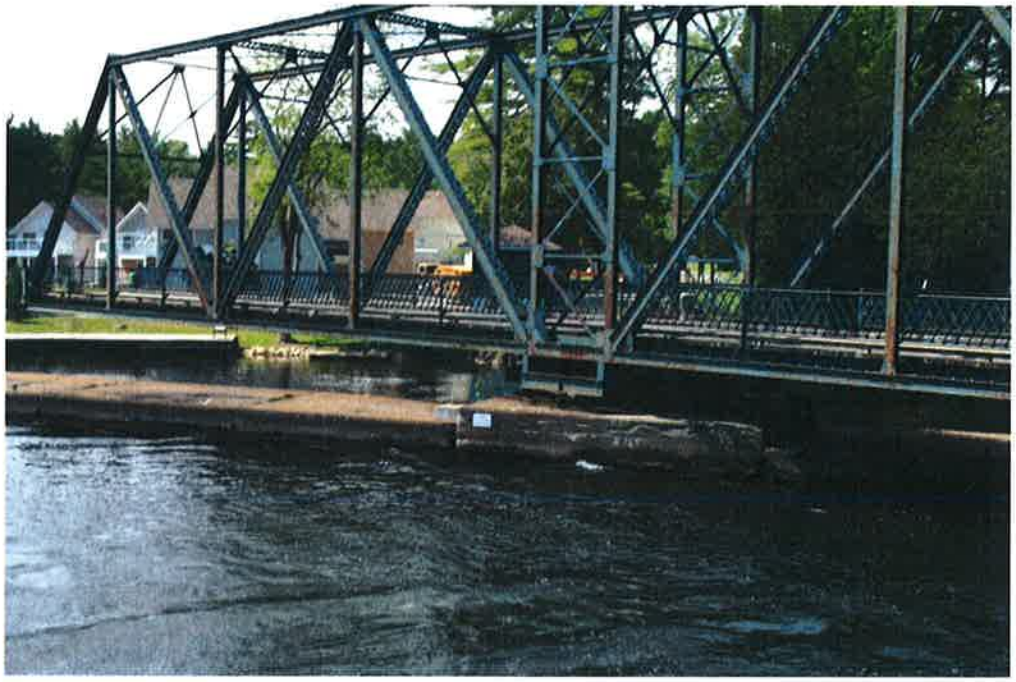


Figure 4-3 View of swing bridge in process of pivoting on centre pier.

Figure 4-4 Overview of southeast wingwall facing north. Note the location of cores C3 (lower) and C4 (upper).

Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-5 Overview of east abutment.

Figure 4-6 Overview of northeast wingwall with locations of cores C5 (upper) and C6 (lower).

Date: March, 2012	Golder Associates Ltd.	Inputted By:	PEB
Project: 11-1111-0118		Checked By:	____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-7 Overview of northwest wingwall.

Figure 4-8 Overview of west abutment.

Date: March, 2012		Golder Associates Ltd.	Inputted By:	PEB
Project: 11-1111-0118			Checked By:	____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-9 Overview of southwest wingwall.

Figure 4-10 Overview of north side of pivot pier.

Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-11 Overview of centre pivot pier facing north.

Figure 4-12 Locations of cores C11 and C12 on north side of pivot pier, facing south.

Date: March, 2012	Golder Associates Ltd.	Inputted By: PEB
Project: 11-1111-0118		Checked By: ____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-13 Overview of south side of pivot pier.

Figure 4-14 Overview of centre pivot pier facing south.

Date:	March, 2012	**Golder Associates Ltd.**	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

Bridge Photographs

Hamlet Bridge
Hamlet, Ontario

Figure 4-15 Location of cores C13 and C14 on south side of pivot pier.

Date:	March, 2012	Golder Associates Ltd.	Inputted By:	PEB
Project:	11-1111-0118		Checked By:	____

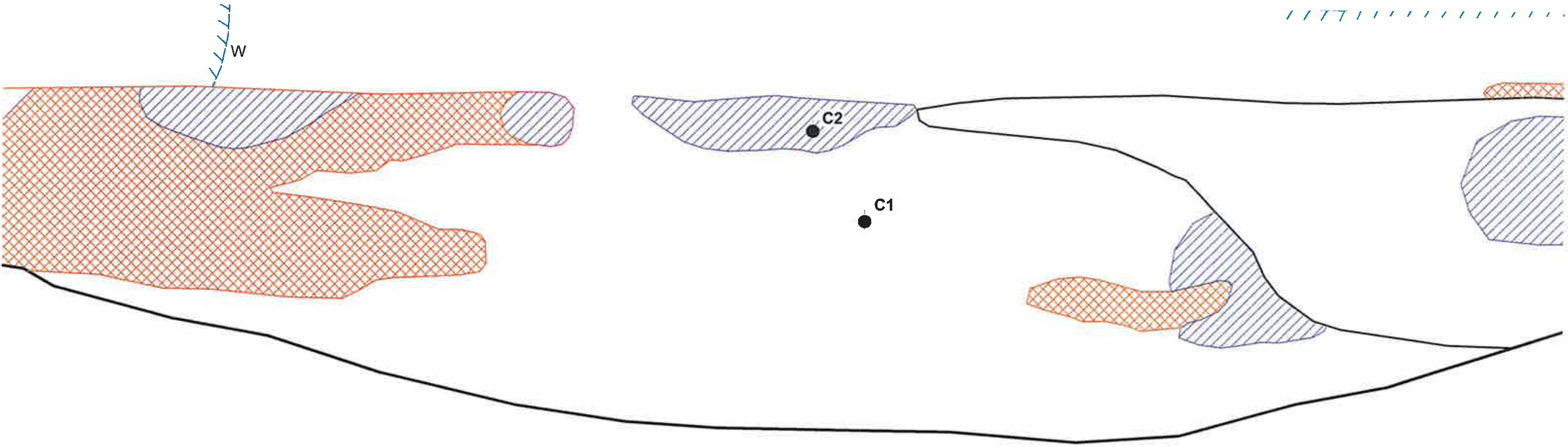
ATTACHMENT 5

DRAWINGS

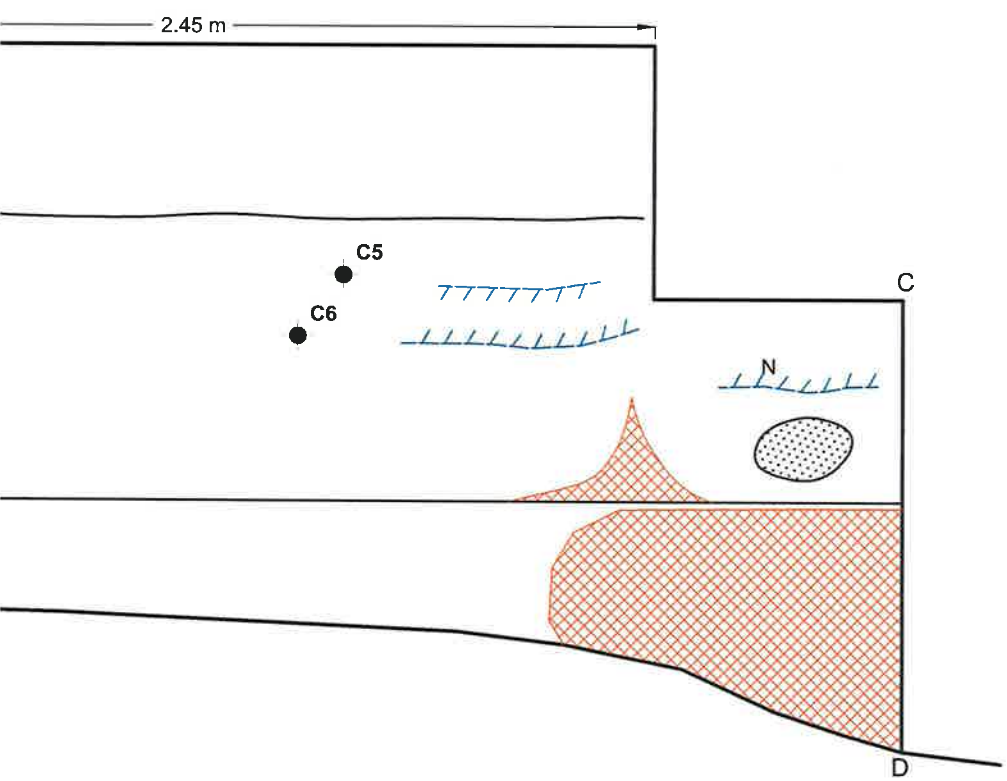
FIGURE 5-1 – EAST ABUTMENT AND WINGWALL DETERIORATION

FIGURE 5-2 – WEST ABUTMENT AND WINGWALL DETERIORATION

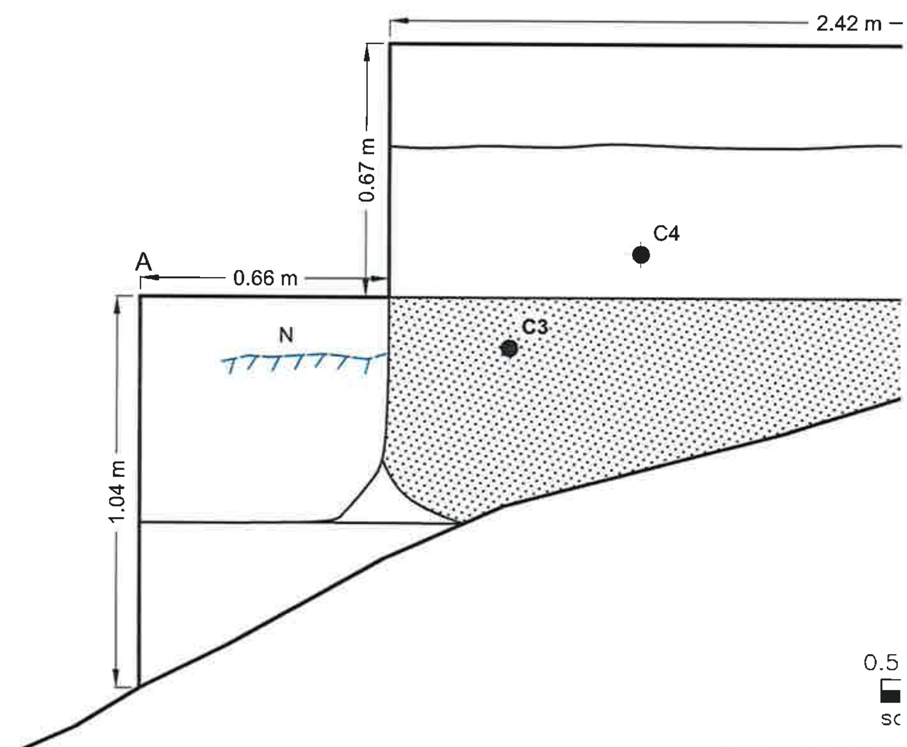
FIGURE 5-2 – CENTER PIVOT PIER DETERIORATION



NORTHEAST WINGWALL



SOUTHEAST WIN



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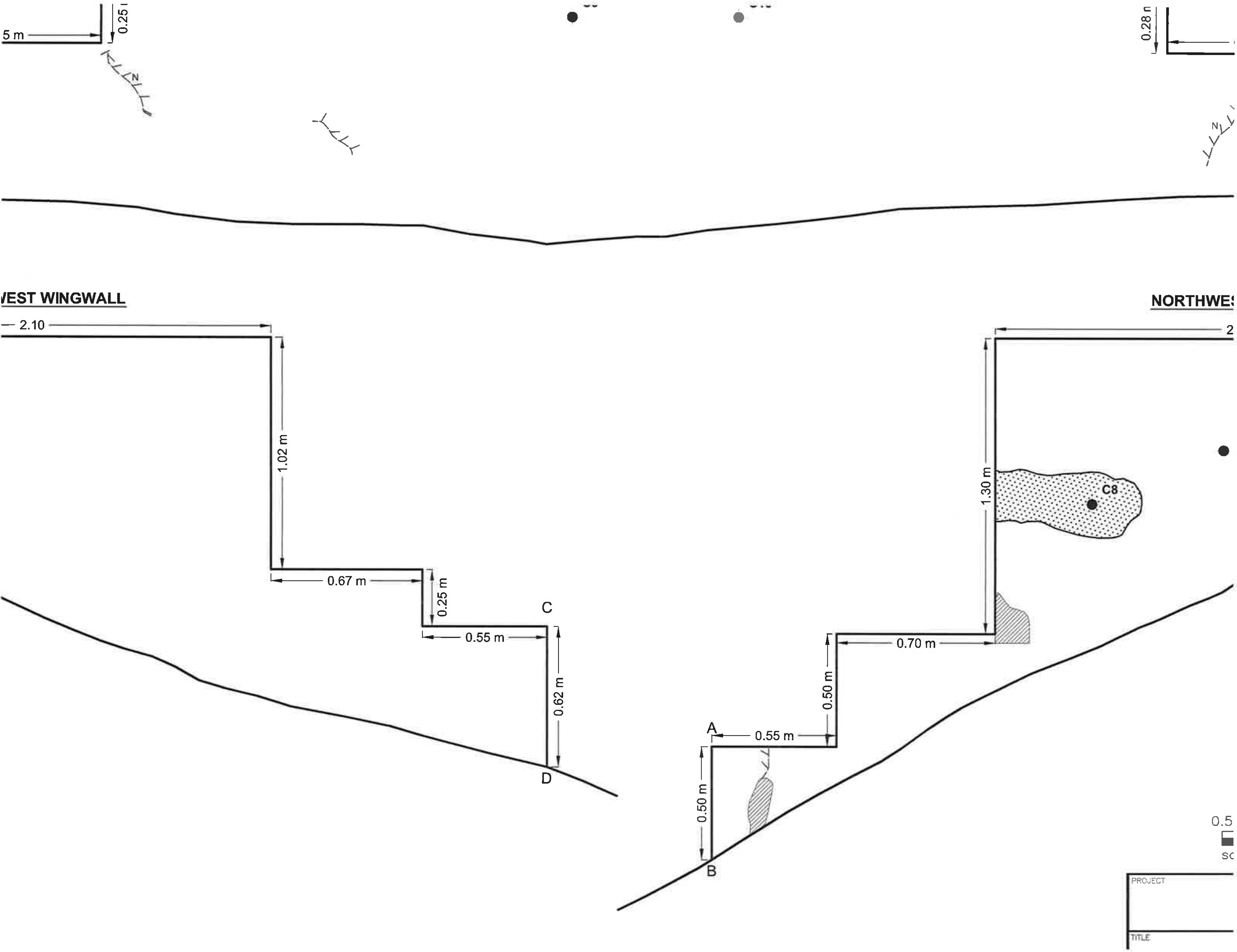
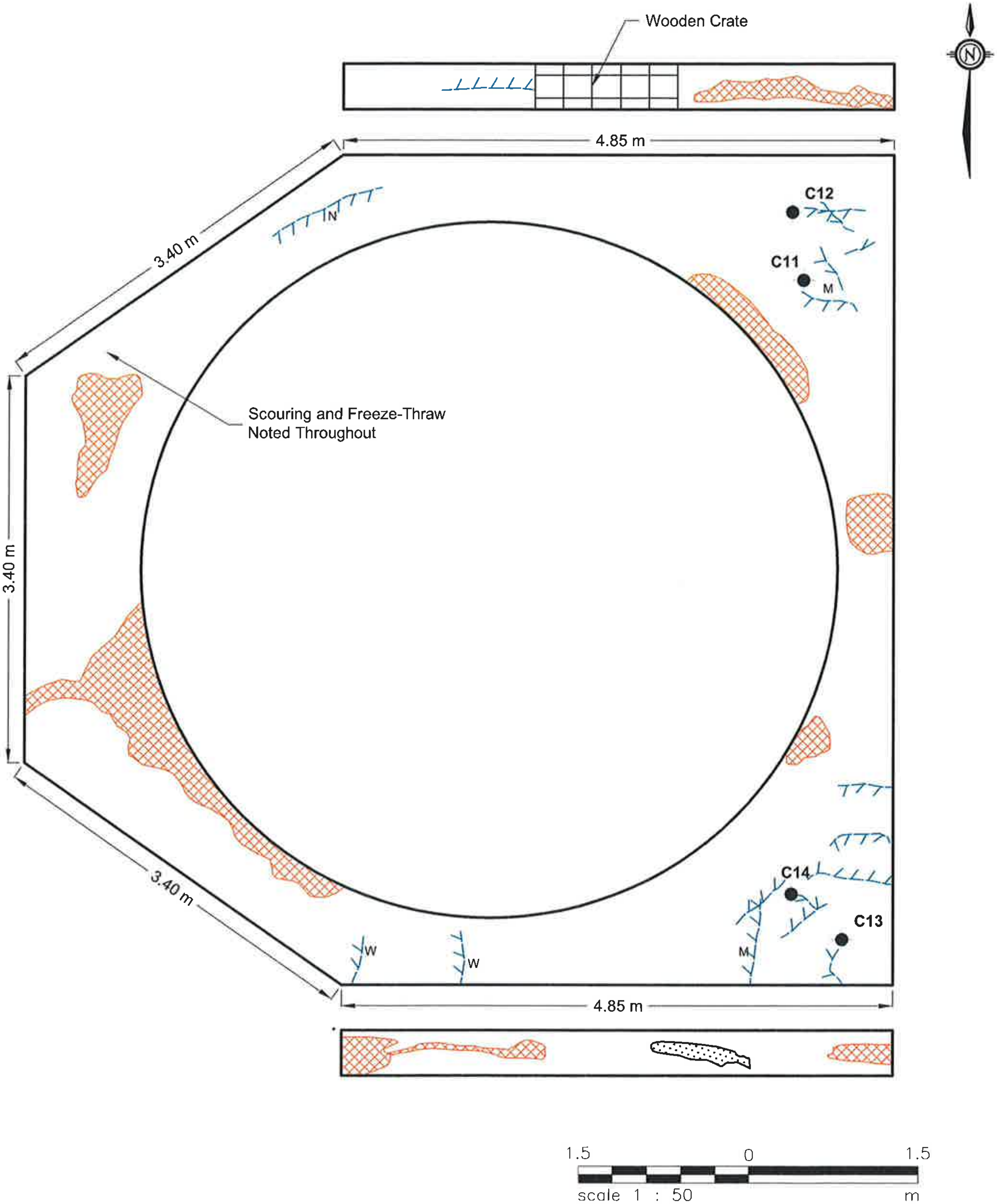
PROJECT
TITLE



5 m
0.25
0.28
VEST WINGWALL
NORTHWES
2.10
2
1.02 m
0.67 m
0.25 m
0.55 m
0.62 m
C
D
1.30 m
C8
0.70 m
0.50 m
0.55 m
0.50 m
A
B
0.5
PROJECT
TITLE



LEGEND

- CONCRETE CORE SAMPLE
- CONCRETE CRACKS STAINED (N=narrow, M=medium, W=wide)
- SPALLING
- SCALING

PROJECT	DELCAN HAMLET BRIDGE HAMLET, ONTARIO			
TITLE	**CENTRE PIVOT PIER DETERIORATION**			
Golder Associates	PROJECT No. 11-1111-0118		FILE No.	AA5-3
	DESIGN		SCALE AS SHOWN	REV.
	CADD	JT	OCT. 2011	**FIGURE 5-3**
	CHECK			
	REVIEW			

Drawing file: N:\CAD\Projects\2011\11-1111-0118 (Delcan, Central Ontario)\-AA-Hamlet Bridge\11111101118AA5-3.dwg Nov 09, 2011 - 3:09pm

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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Golder Associates

APPENDIX I

MECHANICAL INSPECTION PHOTOGRAPHS

Photo M1: The center pivot top plate mounting bolts exhibit moderate corrosion with moderate section loss.

Photo M2: One of six anchor bolts exhibits corrosion and light section loss.

Photo M3: The interior of the pivot girder collects debris and standing water.

Photo M4: There is an impression on top of the rail from the wheel as a result of carrying live load.

Photo M5: The rail support pier is undermined along a significant portion of the rail.

Photo M6: The balance wheel assembly and mounting bolts exhibit moderate corrosion and light section loss.

Photo M7: Northeast End Castor Rest Plate. Slight movement was noted between both end rest plates and the pier. One of the four north rest plate anchor bolts is bent. Also note the heavy wear on the rest plate due to contact with the roller.

Photo M8: The west end castors are no longer utilized to support the swing span. The end lift cylinder body, base plates, and anchor bolts exhibit light corrosion.

Photo M9: North End Lift. The base plate anchor bolts are not properly tightened.

Photo M10: General view of the west locking pin machinery.

Photo M11: The east centering lock pin does not travel far enough to engage the receiver.

Photo M12: General view of the east locking pin.

Photo M13: The east end of travel stop is installed with an energy absorbing pad that is in poor condition.

Photo M14: The east end of travel stop anchor bolts are in poor condition and exhibit evidence that the stop was impacted resulting in the anchors being slightly pulled out.

Photo M15: West end of travel stop. The rest plate anchor bolt heads are cut off and do not secure the rest plate to the pier.

Photo M16: HPU. One hose is abraded at a location near the hose exit from the operator's house.

Photo M17: The flexible hoses that connect the span drive cylinders to the hard piping are abraded.

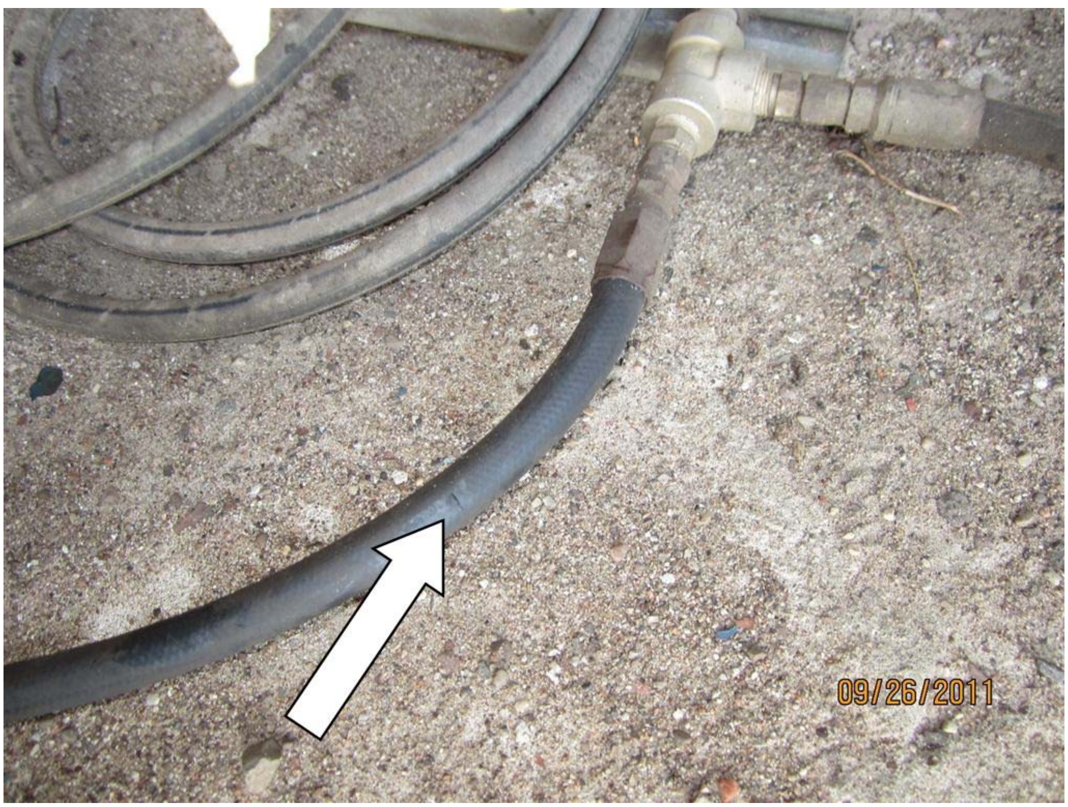


Photo M18: The south cylinder blind end hose is nicked and exhibits blistering.

Photo M19: The blind end flexible hose connecting the HPU to the piping at the pier exhibits a severe bend radius (less than 120 mm radius).

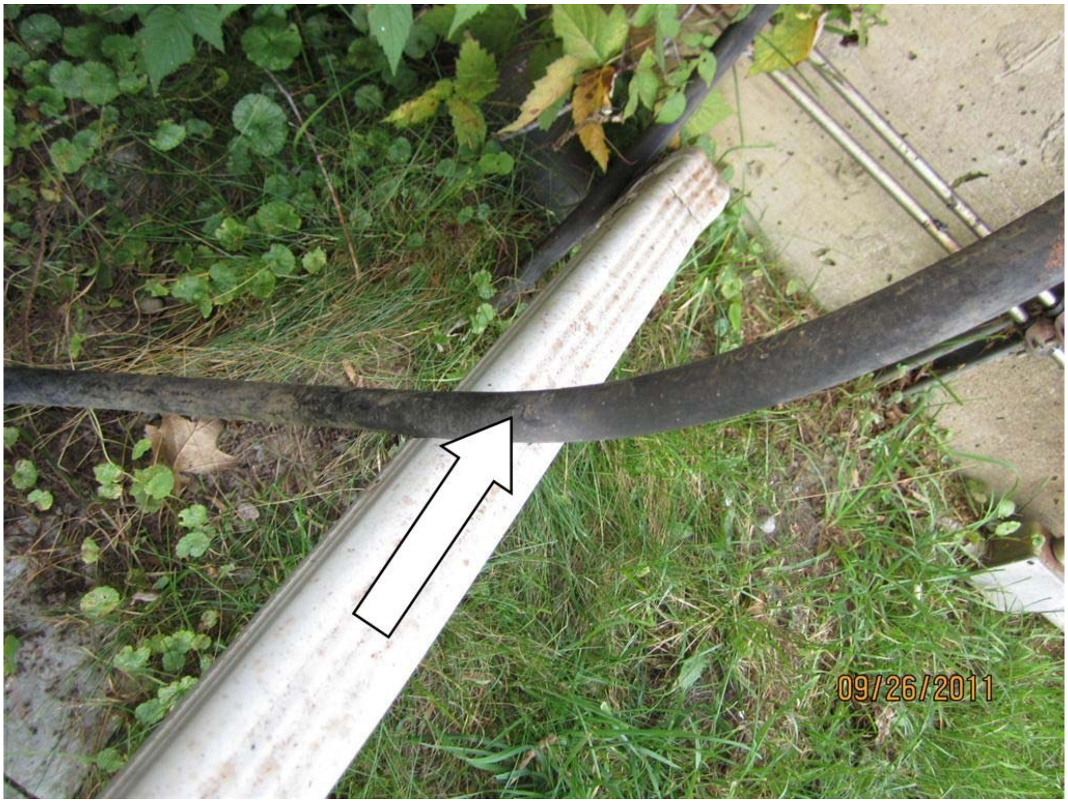


Photo M20: The west locking pin hydraulic cylinder blind end hose (arrow) is damaged.

Photo M21: The blind end clevis and bracket for the span drive cylinder collects debris and exhibits light corrosion.

Photo M22: East Traffic Gate. The gate housings are in fair condition.

Photo M23: West Traffic Gate. The gate arm bearings are in poor condition and appear heavily worn.

APPENDIX J

ELECTRICAL INSPECTION PHOTOGRAPHS

Photo E1: Existing oil filled pole mounted transformer providing 120/240V, single phase service for the bridge operating system. Note the lightning and fused cutouts used to protect the transformer installation.

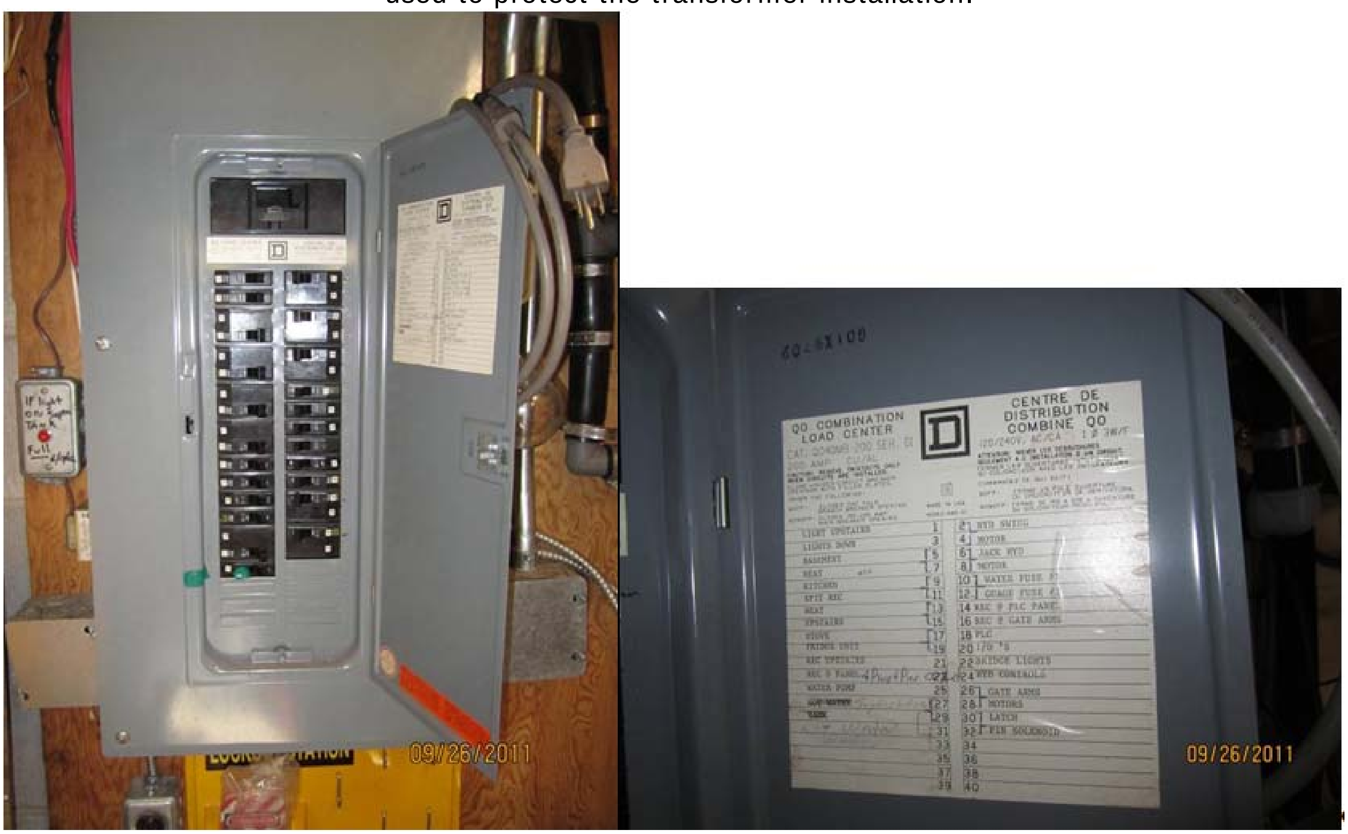


Photo E2: Main Distribution Panelboard. Note the building power and the bridge operating power is fed from this single source.

Photo E3: The swing span hydraulic pump motor, end lift / locking pin hydraulic pump motor, and solenoid valve system was replaced as part of the 1991/1992 rehabilitation and, as can be seen, is in good operational condition having been well installed and housed in the protected area of the bridge control building.

Photo E4: Swing Span Hydraulic Pump Motor. Note the gap in the motor power junction box.

Photo E5: Swing Span Hydraulic Pump Motor Starter. Note the as new condition of the starter (Replaced in 2009).

Photo E6: PLC Controller in the wall mounted enclosure. Note the PLC controller is obsolete.

Photo E7: Bridge Operators Control Console. Note the two pushbutton controls for traffic control and bridge operation and the limited indication lights for status indication. Additionally, note the pushbutton at the lower right hand corner is not labelled.

Photo E8: Bridge End Lift System. Note the corrosion on the limit switch support.

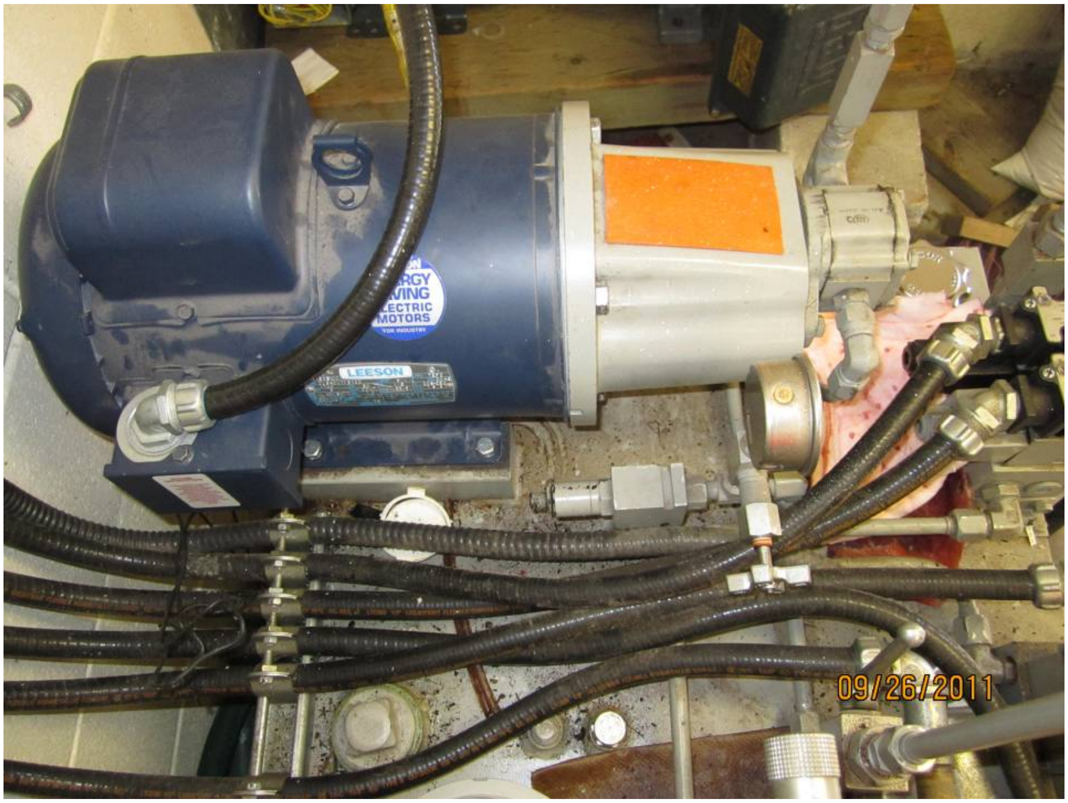


Photo E9: End Lift / Locking Pin Hydraulic Pump Motor. Note the as new condition of the motor.

Photo E10: End Lift / Locking Pin Hydraulic Pump Motor Starter. Note the starter is a single phase starter provided with thermal overload protection.

Photo E11: West Locking Pin. Note the corrosion on the mounting plate.

Photo E12: Typical Bridge Drive System Control Limit Switches. Note the debris on and around the limit switches.

Photo E13: Typical Traffic Signal. Note the good condition of the pole and the signal heads. Also note the warning sign mounted on the signal pole.

Photo E14: West Approach Stop Bar. Note the stop bar is heavily worn.

Photo E15: Typical Traffic Gate. Note the gong is provided on top of the gate enclosure to provide audible warning for the traveling public. Also note the cutout for the hand crank mechanism.

Photo E16: Typical Traffic Gate Equipment. Note the gate is provided with raised and lowered limit switch but no hand crank limit or door switches are provided.

Photo E17: Span Navigation Lights.

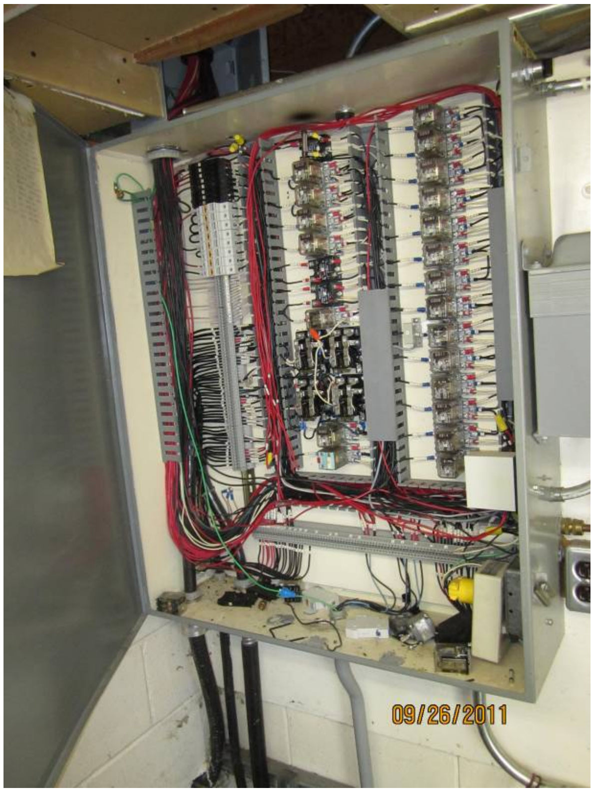


Photo E18: Relay/Contactor Panel. Note the spare parts and wires stored at the bottom of the enclosure. Also note the wires are tagged for ease of troubleshooting.

Photo E19: Center Pier Submarine Cable Junction Box. Note the poor installation of the wires.

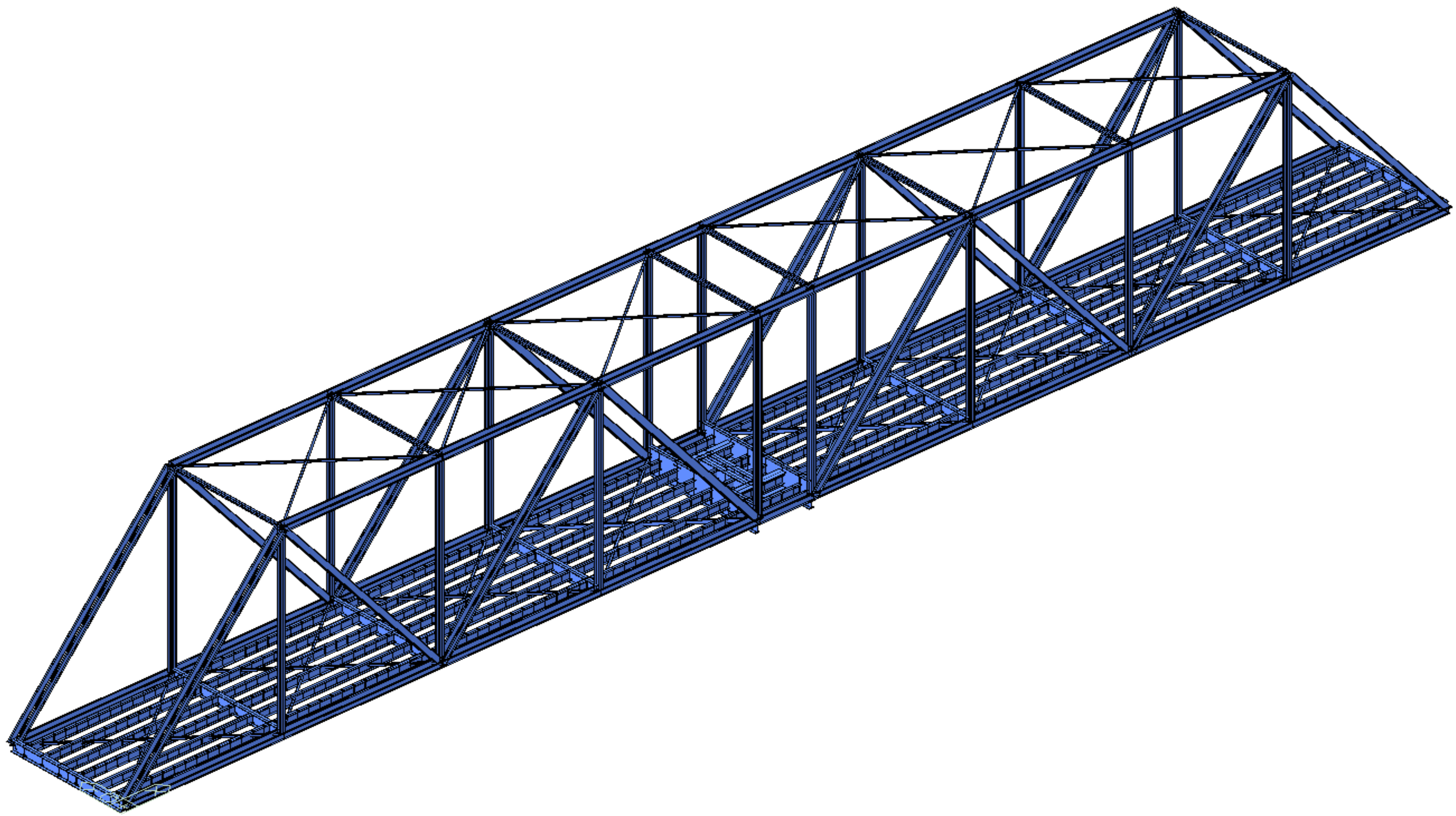
Photo E20: Typical Junction Box. Note the spare or unused wires are not properly terminated.

APPENDIX K

STRUCTURE EVALUATION DATA

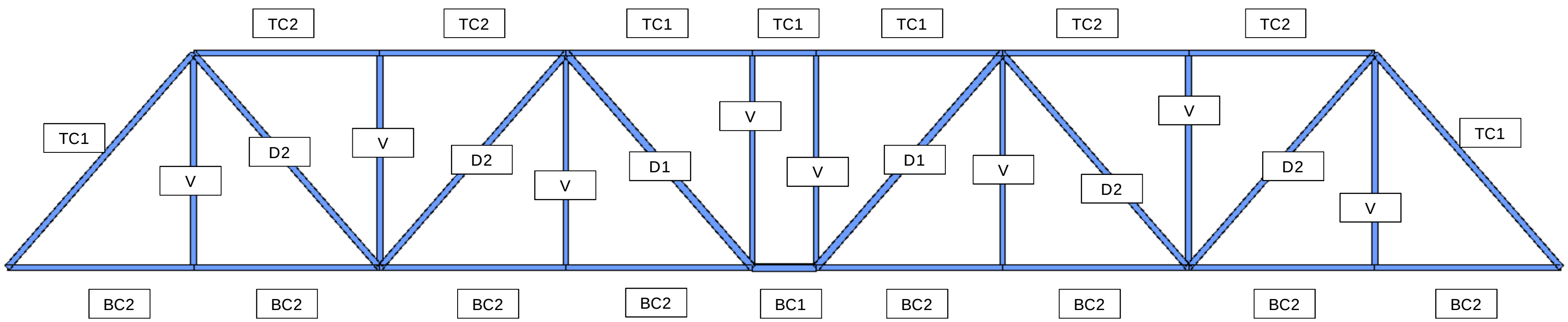
Model Views

3D Model View



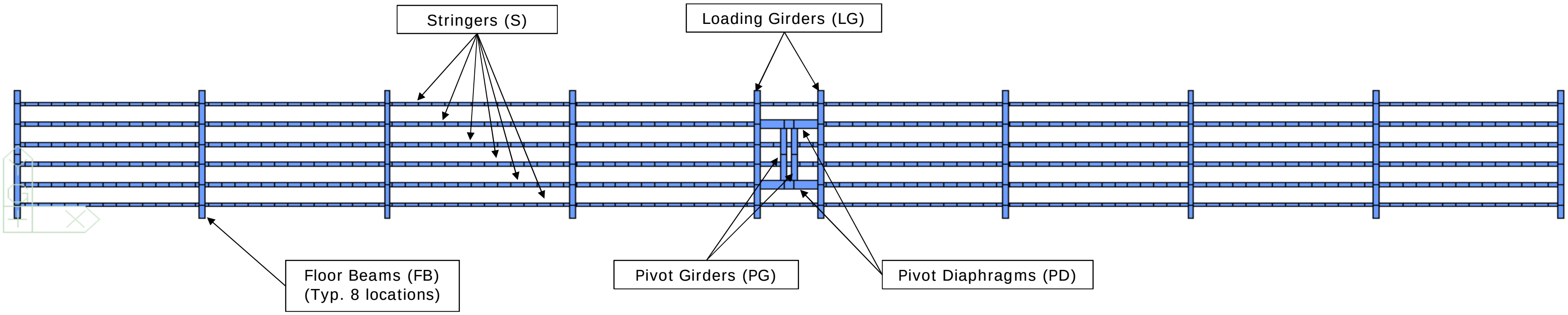
Model Views

Truss Members



Model Views

Floor System



Swing Span Weight

Description	Component	Section	Material Density kg/m^3	Section Area mm^2	Unit Weight kg/m	No.	Length m	Mass kg	Nominal Mass tonnes	Added %	Mass Added tonnes	Total Mass tonnes	Notes
FLOOR SYSTEM													
Floor Beams		I 20 X 72			107.1	8	5.5	4703	4.7	5%	0.24	4.9	tonnes
												48.4	*kN*
												1.10	*kN/m*
Stringers		I 15 X 54			80.4	6	59.7	28805	28.8	5%	1.44	30.2	tonnes
												296.7	*kN*
												0.83	*kN/m*
Loading Girder	Flange Angles	4 - L 6 x 4 x 3/8	7850	9316	73.1	2	5.5	802	0.8	5%	0.04	0.8	tonnes
	Web Plate	PL 16 x 3/8	7850	13548	106.4	2	5.5	1167	1.2	5%	0.06	1.2	tonnes
												20.3	*kN*
												1.85	*kN/m*
Pivot Girder		I 28 x 105			156.3	2	2.9	913	0.9	5%	0.05	1.0	tonnes
												9.4	*kN*
												1.61	*kN/m*
Pivot Diaphragm		I 28 x 165			245.6	2	2.4	1198	1.2	5%	0.06	1.3	tonnes
												12.3	*kN*
												2.53	*kN/m*
											Total floor system mass	**38.2**	**tonnes**
												38242	kg
												375	kN
TRUSSES													
Bottom Chord 1		2-C 12 x 20.7			61.6	2	2.4	300	0.3	5%	0.02	0.3	tonnes
												3.1	*kN*
												0.6	*kN/m*
Bottom Chord 2		2-C 10 x 15.3			45.5	16	7.2	5219	5.2	5%	0.26	5.5	tonnes
												53.8	*kN*
												0.5	*kN/m*
Top Chord 1	Channels	2-C 10 x 20			59.5	2	40.0	4761	4.8	5%	0.24	5.0	tonnes
	Cover Plate	PL 16 x 3/8	7850	3871	30.4	2	40.0	2430	2.4	5%	0.12	2.6	tonnes
												74.1	*kN*
												0.9	*kN/m*
Top Chord 2	Channels	2-C 10 x 15.3			45.5	2	28.7	2609	2.6	5%	0.13	2.7	tonnes
	Cover Plate	PL 16 x 3/8	7850	3871	30.4	2	28.7	1741	1.7	5%	0.09	1.8	tonnes
												44.8	*kN*
												0.8	*kN/m*
Diagonals 1		2-C 12 x 30			89.3	2	23.2	4148	4.1	5%	0.21	4.4	tonnes
												42.7	*kN*
												0.9	*kN/m*
Diagonals 2		2-C 10 x 15.3			45.5	2	46.5	4231	4.2	5%	0.21	4.4	tonnes
												43.6	*kN*
												0.5	*kN/m*
Verticals		HP 8 x 34.3			51.0	16	9.1	7468	7.5	5%	0.37	7.8	tonnes
												76.9	*kN*
												0.5	*kN/m*
											Total trusses mass	**30.2**	**tonnes**
												30173	kg
												296	kN
BRACING													
Top End Ties		2-L 6 x 3-1/2 x 3/8	7850	4414	34.6	2	5.5	380	0.4	5%	0.02	0.4	tonnes
												3.9	*kN*
												0.4	*kN/m*
Top Ties		4-L 3 x 3 x 5/16	7850	4596	36.1	6	5.5	1188	1.2	5%	0.06	1.2	tonnes
												12.2	*kN*
												0.4	*kN/m*
Vertical Brace 1		2-L 31/2 x 31/2 x 3/8	7850	3200	25.1	2	5.5	276	0.3	5%	0.01	0.3	tonnes
												2.8	*kN*
												0.3	*kN/m*
Vertical Brace 2		2-L 3 x 3 x 5/16	7850	2298	18.0	2	24.4	879	0.9	5%	0.04	0.9	tonnes
												9.1	*kN*
												0.2	*kN/m*
Vertical Brace 3		2-L 3 x 3 x 5/16	7850	2298	18.0	6	15.3	1659	1.7	5%	0.08	1.7	tonnes
												17.1	*kN*
												0.2	*kN/m*
Vertical Brace 4		1-L 3 x 3 x 5/16	7850	1149	9.0	2	34.1	614	0.6	5%	0.03	0.6	tonnes
												6.3	*kN*
												0.1	*kN/m*
Top Brace		1-L 3 x 3 x 5/16	7850	1149	9.0	1	125.4	1131	1.1	5%	0.06	1.2	tonnes
												11.6	*kN*
												0.1	*kN/m*
Bottom Brace 1		1-L 3 x 3 x 3/8	7850	1362	10.7	1	86.9	929	0.9	5%	0.05	1.0	tonnes
												9.6	*kN*

Swing Span Weight

Description	Component	Section	Material Density kg/m^3	Section Area mm^2	Unit Weight kg/m	No.	Length m	Mass kg	Nominal Mass tonnes	Added %	Mass Added tonnes	Total Mass tonnes	Notes
												0.1	*kN/m*
Bottom Brace 2		1-L 31/2 x 31/2 x 3/8	7850	1600	12.6	1	36.1	453	0.5	5%	0.02	0.5	tonnes
												4.7	*kN*
												0.1	*kN/m*
Bottom Brace 3		1-L 6 x 31/2 x 3/8	7850	2207	17.3	1	36.1	625	0.6	5%	0.03	0.7	tonnes
												6.4	*kN*
												0.2	*kN/m*
										Total bracing mass		**8.5**	**tonnes**
												8541	kg
												84	kN
RAILINGS													
Railing	Top Rail	1-T 4 x 21/2 x 5/16	7850	1310	10.3	2	59.7	1229	1.2	20%	0.25	1.5	tonnes
	Bottom Rail	1-L 21/2 x 2 x 1/4	7851	683	5.4	2	59.7	641	0.6	20%	0.13	0.8	tonnes
	Lattice	Assumed 20% of top and bottom rail weight.											
	Post	1-L 31/2 x 3 x 5/16	7851	1245	9.8	16	1.5	234	0.2	20%	0.05	0.3	tonnes
												24.8	*kN*
										Total railings mass		**2.5**	**tonnes**
												2524	kg
												25	kN
TIMBER DECK													
Timber Deck		2 x 4 in.	612	434050	266	1	61.0	16204	16.2	0%	0.00	16.2	tonnes
												159.0	kN
Running Boards		2 x 50 in.	612	62500	38	2	61.0	4667	4.7	0%	0.00	4.7	tonnes
												45.8	kN
										Total timber mass		**20.9**	**tonnes**
												20870	kg
												205	kN
										TOTAL BRIDGE WEIGHT		**100**	**tonnes**
												984.4	*kN*

Notes
- Steel member section areas are without consideration of section loss
- Lengths and sizes are not exact

Level 1 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_f (kN)	M_f (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Loading Girders	LG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	103	146	27	33	141	194	176	249	1207	1540	4.07	3.63
Pivot Girders	PG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	118	152	30	39	160	207	559	722	1036	969	1.05	0.71
Pivot Diaphragms	PD	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	116	117	30	31	158	161	340	231	1373	1714	2.40	4.51
Floor Beams	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	20	28	13	20	36	52	245	341	580	522	1.56	0.97
Stringers	S	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	3.1	5.5	2.6	4.6	6	11	98	152	402	298	2.99	1.40
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	---	0	---	0	NA	0	---	23	---	23.8	---	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords/End Posts	TC1	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	72	20	103	376	1035	1.46
	TC2	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	48	13.3	69	297	819	1.49
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	11	0	12	73	305	2.46
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	349	1155	1.78

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Bottom Chords	BC2	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	43	12.1	60	222	1148	3.29
Verticals	V	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	32	13	48	252	1287	3.46
Diagonals	D2	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	41	12	57	233	1148	3.30

BEARING CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		P (kN)	P (kN)	P_f (kN)	P (kN)	B_r (kN)	
Pivot Girders	PG	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	236	60	332	1117	2750	1.27

CONNECTION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		V (kN)	V (kN)	V_f (kN)	V (kN)	V_r (kN)	
Pivot Girder	PG	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.7	118	30	166	559	1368	1.26
Floor Beams	FB	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	20	13	37	245	550	1.28
Stringers	S	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	3.1	2.6	6.1	98	367	2.73
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	32	13	50	252	873	2.00
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	349	825	1.20
	D2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	41	12	59	233	871	2.14

Level 2 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_f (kN)	M_f (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Loading Girders	LG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	103	146	27	33	141	194	159	226	1207	1540	4.50	4.00
Pivot Girders	PG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	118	152	30	39	160	207	504	651	1036	969	1.17	0.79
Pivot Diaphragms	PD	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	116	117	30	31	158	161	306	202	1373	1714	2.66	5.16
Floor Beams	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	20	28	13	20	36	52	245	341	580	522	1.56	0.97
Stringers	S	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	3.1	5.5	2.6	4.6	6	11	98	152	402	298	2.99	1.40
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0	0	0	0	NA	0		23	---	23.8	NA	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords/End Posts	TC1	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	72	20	103	335	1035	1.64
	TC2	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	48	13.3	69	270	819	1.63
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	11	0	12	66	305	2.72
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	316	1155	1.96

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Bottom Chords	BC2	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	43	12.1	60	200	1148	3.65
Verticals	V	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	32	13	48	252	1287	3.46
Diagonals	D2	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	41	12	57	214	1148	3.59

BEARING CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		P (kN)	P (kN)	P_f (kN)	P (kN)	B_r (kN)	
Pivot Girders	PG	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	236	60	332	1007	2750	1.41

CONNECTION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		V (kN)	V (kN)	V_f (kN)	V (kN)	V_r (kN)	
Floor Beams	FB	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	20	13	37	245	550	1.28
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	32	13	50	252	873	2.00
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	316	825	1.32
	D2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	41	12	59	214	871	2.33

Level 3 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_f (kN)	M_f (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Loading Girders	LG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	103	146	27	33	141	194	128	180	1207	1540	5.59	5.02
Pivot Girders	PG	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	118	152	30	39	160	207	394	509	1036	969	1.49	1.01
Pivot Diaphragms	PD	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	116	117	30	31	158	161	244	156	1373	1714	3.34	6.68
Floor Beam	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	20	28	13	20	36	52	245	341	580	522	1.56	0.97
Stringers	S	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	3.1	5.5	2.6	4.6	6	11	98	152	402	298	2.99	1.40
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0	0	0	0	NA	0		23	---	23.8	NA	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords/End Posts	TC1	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	72	20	103	263	1035	2.08
	TC2	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	48	13.3	69	210	819	2.10
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	11	0	12	52	305	3.46
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	250	1155	2.48

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Bottom Chords	BC2	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	43	12.1	60	157	1148	4.65
Verticals	V	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	32	13	48	252	1287	3.46
Diagonals	D2	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	41	12	57	178	1148	4.32

BEARING CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		P (kN)	P (kN)	P_f (kN)	P (kN)	B_r (kN)	
Pivot Girders	PG	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.70	236	60	332	788	2750	1.81

CONNECTION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfact. DL		Fact. DL	Unfact. LL	Fact. Resist.	LL Cap. Factor (F)
										D1	D2				
						D1	D2	D3		V (kN)	V (kN)	V_f (kN)	V (kN)	V_r (kN)	
Floor Beams	FB	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	20	13	37	245	550	1.28
Verticals	V	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	32	13	50	252	873	2.00
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	102	28	144	250	825	1.67
	D2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	41	12	59	178	871	2.80

Timber Deck - Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

b	Section width	=	38	mm	= 1.5"
d	Section height	=	89	mm	= 3.5"
S_x	Elastic section modulus	=	5.02E+04	mm^3	= 1/6 * bh^2

FLEXURAL RESISTANCE (Cl. 9.6.1)

ϕ	Resistance factor (flexure)	=	0.90		Table 9.1
k_d	Load duration factor	=	1.00		Cl. 9.5.3, dead and live loads
k_{ls}	Lateral stability factor	=	1.00		Table 9.5
k_m	Load-sharing factor	=	1.40		Cl. 9.5.6
k_{sb}	Size effect factor	=	1.70		Table 9.4
f_{bu}	Bending at extreme fibre	=	8.40		Table 9.12, for SPF 1/2
M_r	Factored moment resistance	=	0.90	kN.m	= $\phi\ k_d\ k_{ls}\ k_m\ k_{sb}\ f_{bu}\ S$
	(x 1000/38 for a 1 metre wide strip)	=	23.8	kNm	

Section S - Properties

Member Type/Location:	**Stringer - Section S**
Member Description:	**I 15 X 54**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	178	19	371.5	3387	102432	1258219	181	110933804	111036236
Web	1	9	343	190.5	3141	30776320	598354	0	0	30776320
Bottom Flange	1	178	19	9.5	3387	102432	32262	181	110933804	111036236
					9915		1888835			252848792

Overall Section Depth:	381	mm	Section Modulus Top (S_x):	1.33E+06	mm^3
Centroid Distance From Bottom of Section:	191	mm	Section Modulus Bottom (S_x) :	1.33E+06	mm^3
Section Moment of Inertia (I_x):	2.53E+08	mm^4	Radius of Gyration (r_x) :	159.7	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	178	19	88.9	3387	8922961	301112	0.0	0	8922961
Web	1	9	343	88.9	3141	21962	279232	0.0	0	21962
Bottom Flange	1	178	19	88.9	3387	8922961	301112	0.0	0	8922961
					9915		881456			17867884

Overall Section Width:	178	mm	Section Modulus Top (S_y):	2.01E+05	mm^3
Centroid Distance From Right Side of Section:	89	mm	Section Modulus Bottom (S_y):	2.01E+05	mm^3
Section Moment of Inertia (I_y):	1.79E+07	mm^4	Radius of Gyration (r_y) :	42.5	mm

STRONG AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	178	19	371.5	3387	3387	181	0	0	612979
Web	1	9	343	190.5	3141	1570	86	1570	86	269259
Bottom Flange	1	178	19	9.5	3387	0	0	3387	181	612979
					9915	4958		4958		1495216

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	191	mm
Overall Section Depth:	381	mm
Plastic Section Modulus Z_x:	1.50E+06	mm^3

WEAK AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	177.8	19.1	88.9	3387	1694	44	1694	44	150556
Web	1	9.2	342.9	88.9	3141	1570	2	1570	2	7193
Bottom Flange	1	177.8	19.1	88.9	3387	1694	44	1694	44	150556
					9915	4958		4958		308305

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	89	mm
Overall Section Depth:	178	mm
Plastic Section Modulus Z_y:	3.08E+05	mm^3

Section S - Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	
b_1	Top flange width	=	178	mm	= 7"
t_1	Top flange thickness	=	19	mm	= 0.75"
h	Web height	=	343	mm	
w	Web width	=	9	mm	= 0.4" - 1 mm
b_2	Bottom flange width	=	178	mm	= 7"
t_2	Bottom flange thickness	=	19	mm	= 0.75"
Z_x	Plastic modulus	=	1.50E+06	mm^3	
S_x	Elastic section modulus (top)	=	1.33E+06	mm^3	
S_x	Elastic section modulus (bottom)	=	1.33E+06	mm^3	
I_y	Moment of inertia	=	1.79E+07	mm^4	
J	Torsional constant	=	9.07E+05	mm^4	Figure C10.2
d_1		=	362	mm	$h + t_1/2 + t_2/2$
C_w	Warping constant	=	5.84E+11	mm^6	Figure C10.2
β_x	Coefficient of monosymmetry	=	0.0		Figure C10.2
A_w	Shear area	=	3490	mm^2	

SECTION CLASSIFICATION (Cl. 10.9.2)

Top Flange in Compression

b	Half flange width	=	89	mm	
t	Flange thickness	=	19	mm	
b/t		=	4.7		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Top flange class	=	Class 1		

Web

h	Web height	=	343	mm	
w	Web width	=	9	mm	
h/w		=	37.4		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	75.9		$= 1100 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1.

FACTORED MOMENT RESISTANCE (Cl. 10.10.2)

ϕ_s	Steel resistance factor (bending)	=	0.95		Cl. 10.5.7
F_y	Yield strength	=	210	MPa	

Laterally Supported Members (Class 1 or 2 Sections)

M_p	Plastic moment	=	314	kN.m	Cl. 10.10.2.2
M_r	Factored moment resistance	=	298	kN.m	

Section S - Resistance

SHEAR RESISTANCE (Cl. 10.10.5.1)

ϕ_s	Steel resistance factor (shear)	=	0.95		Cl. 10.5.7
A_w	Shear area	=	3490	mm^2	
k_v	shear buckling coefficient	=	5.34		
h/w		=	37.43		
	First limit	=	80.1		= 502 sqrt (kv/Fy)
F_{cr}	Shear buckling stress	=	121	MPa	= 0.577 Fy
F_t	Tension field component	=	0	MPa	
F_s	Ultimate shear stress, $F_{cr} + F_t$	=	121	MPa	
V_r	Factored shear resistance	=	402	kN	$= \phi_s A_w F_s$

RIVETED CONNECTION (14.14.1.4)

Rivets in Shear (14.14.1.4.2)

ϕ_{mc}	Resistance factor	=	0.67		
t	Thickness of web	=	9	mm	
n	Number of rivets	=	4		
e	Edge distance	=	50	mm	
F_u	Tensile strength	=	320	MPa	Cl. 14.7.4.2
d	Diameter of rivet	=	19.1	mm	
B_r	Factored bearing resistance	=	393	kN	$= \min(\phi_{mc} tneF_u, 3\phi_{mc} tndF_u)$
ϕ_r	Resistance factor	=	0.67		
n	Number of rivets	=	4		
m	Number of shear planes	=	2		
A_r	Area of rivet	=	285	mm^2	
F_u	Tensile strength of rivet steel	=	320	MPa	Cl. 14.7.4.6
V_r	Factored shear resistance	=	367	kN	$= 0.75\phi_r nmA_r F_u$
	Governing Resistance	=	367	kN	$= \min(B_r, V_r)$

Section FB Properties

Member Type/Location:	**Floor Beam - Section FB**
Member Description:	**I 20 x 72**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	222	20	498.2	4347	138559	2165650	240	251256426	251394985
Web	1	10	470	253.5	4662	85780772	1181865	4	86135	85866907
Bottom Flange	1	222	19	9.3	4125	118373	38271	249	254737662	254856035
					13133		3385786			592117927

Overall Section Depth:	508	mm	Section Modulus Top (S_x):	2.37E+06	mm^3
Centroid Distance From Bottom of Section:	258	mm	Section Modulus Bottom (S_x) :	2.30E+06	mm^3
Section Moment of Inertia (I_x):	5.92E+08	mm^4	Radius of Gyration (r_x) :	212.3	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	222	20	111.1	4347	17892396	483034	0.0	0	17892396
Web	1	10	470	111.1	4662	38248	518086	0.0	0	38248
Bottom Flange	1	222	19	111.1	4125	16977558	458337	0.0	0	16977558
					13133		1459457			34908202

Overall Section Width:	222	mm	Section Modulus Top (S_y):	3.14E+05	mm^3
Centroid Distance From Right Side of Section:	111	mm	Section Modulus Bottom (S_y):	3.14E+05	mm^3
Section Moment of Inertia (I_y):	3.49E+07	mm^4	Radius of Gyration (r_y) :	51.6	mm

STRONG AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	222	20	498.2	4347	4347	234	0	0	1015062
Web	1	10	470	253.5	4662	2220	112	2442	123	548917
Bottom Flange	1	222	19	9.3	4125	0	0	4125	255	1053487
					13133	6567		6567		2617466

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	265	mm
Overall Section Depth:	508	mm
Plastic Section Modulus Z_x:	2.62E+06	mm^3

WEAK AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	222.3	19.6	111.1	4347	2173	56	2173	56	241517
Web	1	9.9	469.9	111.1	4662	2331	2	2331	2	11565
Bottom Flange	1	222.3	18.6	111.1	4125	2062	56	2062	56	229168
					13133	6567		6567		482250

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	111	mm
Overall Section Depth:	222	mm
Plastic Section Modulus Z_y:	4.82E+05	mm^3

Section FB - Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	
b_1	Top flange width	=	222	mm	= 8.75"
t_1	Top flange thickness	=	20	mm	= 0.77"
h	Web height	=	470	mm	
w	Web width	=	10	mm	= 0.43"-1mm
b_2	Bottom flange width	=	222	mm	= 8.75"
t_2	Bottom flange thickness	=	19	mm	= 0.77"-1mm
L	Unsupported length	=	0	mm	
Z_x	Plastic modulus	=	2.62E+06	mm^3	
S_x	Elastic section modulus (top)	=	2.37E+06	mm^3	
S_x	Elastic section modulus (bottom)	=	2.30E+06	mm^3	
I_y	Moment of inertia	=	3.49E+07	mm^4	
J	Torsional constant	=	1.18E+06	mm^4	Figure C10.2
d_1		=	489	mm	$h + t_1/2 + t_2/2$
C_w	Warping constant	=	2.08E+12	mm^6	Figure C10.2
β_x	Coefficient of monosymmetry	=	0.0		Figure C10.2
A_w	Shear area	=	5040	mm^2	

SECTION CLASSIFICATION (Cl. 10.9.2)

Top Flange in Compression

b	Half flange width	=	111	mm	
t	Flange thickness	=	20	mm	
b/t		=	5.7		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Top flange class	=	Class 1		

Web

h	Web height	=	470	mm	
w	Web width	=	10	mm	
h/w		=	47.4		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	75.9		$= 1100 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1.

FACTORED MOMENT RESISTANCE (Cl. 10.10.2)

ϕ_s	Steel resistance factor (bending)	=	0.95		Cl. 10.5.7
F_y	Yield strength	=	210	MPa	

Laterally Supported Members (Class 1 or 2 Sections)

M_p	Plastic moment	=	550	kNm	$= Z_x F_y$
M_r	Factored moment resistance	=	522	kNm	$= \phi s\ Mp$

Section FB - Resistance

SHEAR RESISTANCE (Cl. 10.10.5.1)

ϕ_s	Steel resistance factor (shear)	=	0.95		Cl. 10.5.7
A_w	Shear area	=	5040	mm^2	
k_v	shear buckling coefficient	=	5.34		
h/w		=	47.4		
	First limit	=	80.1		= 502 sqrt (kv/Fy)
F_{cr}	Shear buckling stress	=	121	MPa	= 0.577 Fy
F_t	Tension field component	=	0	MPa	
F_s	Ultimate shear stress, $F_{cr} + F_t$	=	121	MPa	
V_r	Factored shear resistance	=	580	kN	$= \phi_s A_w F_s$

RIVETED CONNECTION (14.14.1.4)

Rivets in Shear (14.14.1.4.2)

ϕ_{mc}	Resistance factor	=	0.67		
t	Thickness of web	=	10	mm	
n	Number of rivets	=	6		
e	Edge distance	=	50.8	mm	
F_u	Tensile strength	=	320	MPa	Cl. 14.7.4.2
d	Diameter of rivet	=	19.1	mm	
B_r	Factored bearing resistance	=	648	kN	$= \min (\phi_{mc} tneF_u, 3\phi_{mc} tndF_u)$
ϕ_r	Resistance factor	=	0.67		
n	Number of rivets	=	6		
m	Number of shear planes	=	2		
A_r	Area of rivet	=	285	mm^2	
F_u	Tensile strength of rivet steel	=	320	MPa	Cl. 14.7.4.6
V_r	Factored shear resistance	=	550	kN	$= 0.75\phi_r nmA_r F_u$
	Governing Resistance	=	550	kN	$= \min(B_r, V_r)$

Section PG Prop

Member Type/Location: **Pivot Girder - Section PG**
Member Description: **I 28 X 108**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	254	21	677.8	5355	198330	3629497	330	581406313	581604643
Web	1	13	647	343.7	8466	295479197	2909410	5	180706	295659903
Bottom Flange	1	254	20	10.0	5101	171425	51217	338	583598353	583769778
					18921		6590125			1461034323

Overall Section Depth: 688 mm
Centroid Distance From Bottom of Section: 348 mm
Section Moment of Inertia (I_x): 1.46E+09 mm^4

Section Modulus Top (S_x): 4.30E+06 mm^3
Section Modulus Bottom (S_x) : 4.19E+06 mm^3
Radius of Gyration (r_x) : 277.9 mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	254	21	127.0	5355	28789340	680063	0.0	0	28789340
Web	1	13	647	127.0	8466	120716	1075145	0.0	0	120716
Bottom Flange	1	254	20	127.0	5101	27423752	647805	0.0	0	27423752
					18921		2403013			56333808

Overall Section Width: 254 mm
Centroid Distance From Right Side of Section: 127 mm
Section Moment of Inertia (I_y): 5.63E+07 mm^4

Section Modulus Top (S_y): 4.44E+05 mm^3
Section Modulus Bottom (S_y): 4.44E+05 mm^3
Radius of Gyration (r_y) : 54.6 mm

STRONG AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	254	21	677.8	5355	5355	324	0	0	1737215
Web	1	13	647	343.7	8466	4106	157	4360	167	1370934
Bottom Flange	1	254	20	10.0	5101	0	0	5101	343	1751307
					18921	9461		9461		4859455

Iterate centroid so difference is zero
Area Difference: 0.0 mm^2
Plastic Centroid Distance From Bottom of Section: 353 mm
Overall Section Depth: 688 mm
Plastic Section Modulus Z_x: 4.86E+06 mm^3

WEAK AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	254.0	21.1	127.0	5355	2677	64	2677	64	340032
Web	1	13.1	647.2	127.0	8466	4233	3	4233	3	27685
Bottom Flange	1	254.0	20.1	127.0	5101	2550	64	2550	64	323903
					18921	9461		9461		691619

Iterate centroid so difference is zero
Area Difference: 0.0 mm^2
Plastic Centroid Distance From Bottom of Section: 127 mm
Overall Section Depth: 254 mm
Plastic Section Modulus Z_y: 6.92E+05 mm^3

Section PG - Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	
b_1	Top flange width	=	254	mm	
t_1	Top flange thickness	=	21.1	mm	
h	Web height	=	647	mm	
w	Web width	=	13.1	mm	
b_2	Bottom flange width	=	254	mm	
t_2	Bottom flange thickness	=	20.1	mm	
L	Unsupported length	=	1300	mm	
Z_x	Plastic modulus	=	4.86E+06	mm^3	
S_x	Elastic section modulus (top)	=	4.30E+06	mm^3	
S_x	Elastic section modulus (bottom)	=	4.19E+06	mm^3	
I_y	Moment of inertia	=	5.63E+07	mm^4	
J	Torsional constant	=	1.96E+06	mm^4	Figure C10.2
d_1		=	668	mm	$h + t_1/2 + t_2/2$
C_w	Warping constant	=	6.26E+12	mm^6	Figure C10.2
β_x	Coefficient of monosymmetry	=	0.0		Doubly symmetric
A_w	Shear area	=	9004	mm^2	

SECTION CLASSIFICATION (Cl. 10.9.2)

Top Flange in Compression

b	Half flange width	=	127	mm	
t	Flange thickness	=	21	mm	
b/t		=	6.0		
F_y	Yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Top flange class	=	Class 1		

Bottom Flange in Compression

b	Half flange width	=	127	mm	Cl. 10.9.2.2
t	Flange thickness	=	20	mm	
b/t		=	6.3		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Bottom flange class	=	Class 1		

Web

h	Web height	=	647	mm	
w	Web width	=	13.1	mm	
h/w		=	49.5		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	75.9		$= 1100 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1.

Section PG - Resistance

FACTORED MOMENT RESISTANCE, CLASS 1 OR SECTIONS (Cl. 10.10.2.3)

ϕ_s	Steel resistance factor (bending)	=	0.95		Cl. 10.5.7
F_y	Yield strength	=	210	MPa	
ω_2	Moment gradient coefficient	=	1.74		See clause for formula; From DL analysis
B_1	Geometric coefficient	=	0.00		See clause for formula
B_2	Geometric coefficient	=	48.4		See clause for formula
M_u	Critical elastic moment	=	38565	kN.m	

Laterally Unsupported Members

M_p	Plastic moment	=	1020	kN.m	$= \phi s\ Z_x\ F_y$
	0.67 Mp	=	684	kN.m	
	Mu > 0.67 Mp				
M_r	Factored moment resistance	=	969	kN.m	Cl. 10.10.2.3

SHEAR RESISTANCE (Cl. 10.10.5.1)

ϕ_s	Steel resistance factor (shear)	=	0.95		Cl. 10.5.7
A_w	Shear area	=	9004	mm^2	
a	Stifffener spacing	=	650	mm	
a/h		=	1.004		
k_v	shear buckling coefficient	=	9.31		See clause for formula
h/w		=	49.5		
	First limit	=	105.7		= 502 sqrt (kv / Fy)
	Second Limit	=	130.7		= 621 sqrt (kv / Fy)
F_{cr}	Shear buckling stress	=	121	MPa	See clause for formula
F_t	Tension field component	=	0	MPa	See clause for formula
F_s	Ultimate shear stress, $F_{cr} + F_t$	=	121	MPa	
V_r	Factored shear resistance	=	1036	kN	$= \phi_s\ A_w\ F_s$

BEARING RESISTANCE (10.10.8)

Web Crippling and Yielding (Cl. 10.10.8.1)

w	Web thickness	=	13	mm	
t	Flange thickness	=	26	mm	Additional plate at pintle bearing
N	Length of bearing	=	533	mm	
F_y	Yield strength	=	210	MPa	
	Location > member depth from end?	=	Yes		
	Not at Member End (> Depth From End)				
ϕ_{bi}	Resistance factor	=	0.80		
B_r	Fact. web compressive resistance (i)	=	1739	kN	$= \phi_{bi}\ w\ (N + 10t)\ F_y$
B_r	Fact. web compressive resistance (ii)	=	1286	kN	$= 1.45\phi_{bi}\ w^2\ \mathrm{sqrt}(F_v E_s)$
	Governing resistance	=	1286	kN	Not including stiffeners

Section PG - Resistance

Bearing Stiffeners

In Bearing (10.10.8.2)

ϕ_s	Resistance factor	=	0.90		
A_s	Area of stiffener in contact with flange	=	9700	mm^2	Parts of 4-C15 x 33.9
F_y	Yield strength of flange or stiffener	=	210	MPa	
B_r	Factored bearing resistance of bearing stiffeners	=	2750	kN	$= 1.50\ \phi_s\ A_s\ F_y$

As Columns

ϕs	Steel resistance factor	=	0.90		For compression
L	Effective length kL	=	550	mm	
r	Minimum radius of gyration	=	37.1	mm	From properties sheet
Es	Steel elastic modulus	=	200000	MPa	
λ		=	0.153		$= kL / r \times \text{sqrt}(Fy/ pi^2\ Es)$
A	Section area	=	17230	mm2	From properties sheet
Cr	Fact. compressive resistance	=	3241	kN	$= \phi s\ A\ Fy\ (1+ \lambda^{2n})^{-1/n}$; n = 1.34

RIVETED END CONNECTIONS (14.14.1.4)

Rivets in Shear (14.14.1.4.2)

ϕ_{mc}	Resistance factor	=	0.67		
t	Thickness of web	=	13	mm	
n	Number of rivets	=	15		
e	Edge distance	=	51	mm	
Fu	Tensile strength	=	320	MPa	Cl. 14.7.4.2
d	Diameter of rivet	=	19.0	mm	= 3/4"
B_r	Factored bearing resistance	=	2149	kN	$= \min(\phi_{mc}\ t\ n\ e\ F_u,\ 3\ \phi_{mc}\ t\ n\ d\ F_u)$
ϕ_r	Resistance factor	=	0.67		
n	Number of rivets	=	15		
m	Number of shear planes	=	2		
A_r	Area of rivet	=	284	mm^2	
F_u	Tensile strength of rivet steel	=	320	MPa	Cl. 14.7.4.6
V_r	Factored shear resistance	=	1368	kN	$= 0.75\phi_r n m A_r F_u$
	Governing Resistance	=	1368	kN	$= \min(B_r, V_r)$

Section BC2 - Properties

Member Type/Location:	**Bottom chord - Section BC2**
Member Description:	**2-C10 x 15.3**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flanges	2	66.0	11.1	248.5	1463	14949	363428	121	21579622	21594572
Webs	2	6.1	231.9	127.0	2827	12662567	358995	0	0	12662567
Bottom Flanges	2	66.0	11.1	5.5	1463	14949	8099	121	21579622	21594572
					5752		730522			55851710

Overall Section Depth:	254	mm	Section Modulus Top (S_x):	4.40E+05	mm^3
Centroid Distance From Bottom of Section:	127	mm	Section Modulus Bottom (S_x) :	4.40E+05	mm^3
Section Moment of Inertia (I_x):	5.59E+07	mm^4	Radius of Gyration (r_x) :	98.5	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange 1	1	66.0	11.1	33.0	731	265803	24149	160.0	18727327	18993131
Top Flange 2	1	66.0	11.1	353.1	731	265803	258212	160.0	18727327	18993131
Web 1	1	6.1	231.9	63.0	1413	4377	89031	130.0	23903509	23907886
Web 2	1	6.1	231.9	323.1	1413	4377	456641	130.0	23903509	23907886
Bottom Flange 1	1	66.0	11.1	33.0	731	265803	24149	160.0	18727327	18993131
Bottom Flange 2	1	66.0	11.1	353.1	731	265803	258212	160.0	18727327	18993131
					5752		1110394			123788294

Clear Gap Between Channels	254	mm			
Overall Section Width:	386	mm	Section Modulus Top (S_y):	6.41E+05	mm^3
Centroid Distance From Right Side of Section:	193	mm	Section Modulus Bottom (S_y):	6.41E+05	mm^3
Section Moment of Inertia (I_y):	1.24E+08	mm^4	Radius of Gyration (r_y) :	146.7	mm

Section BC2 - Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
F_u	Tensile strength	=	420	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	Cl. 10.4.2
b_1	Top flange width	=	66	mm	= 2.6"
t_1	Top flange thickness	=	11	mm	= 0.436"
h	Web height	=	232	mm	
w	Web width	=	6.1	mm	= 0.24"
b_2	Bottom flange width	=	66	mm	= 2.6"
t_2	Bottom flange thickness	=	11	mm	= 0.436"
d_1	Clear gap between channels	=	254	mm	= 10"
L	Unsupported length	=	7100	mm	= 23.5 ft
A	Area	=	5752	mm^2	
I_y	Moment of inertia	=	1.24E+08	mm^4	
I_x	Moment of inertia	=	5.59E+07	mm^4	
r_y	Radius of gyration	=	146.7	mm	
r_x	Radius of gyration	=	98.5	mm	

TENSION MEMBERS (10.8)

Slenderness (10.8.1.2)

K_x	Effective length factor	=	1.0		
L_y	Unsupported length	=	7100	mm	
r	Min. radius of gyration	=	98.5	mm	
kL/r	Slenderness ratio	=	72.1		
	Limit	=	200		Okay

Axial Tensile Resistance (10.8.2)

Gross Section Yielding

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	5752	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	1148	kN	$= \phi_s A_g F_y$

Net Section Fracture

Failure through 1 line of rivets in webs at panel point 1 (abutment)

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
n	Number of rivets in webs	=	3		in one line
d	Diameter of rivets in webs	=	19.1	mm	= 3/4"
	Size of hole	=	20.6	mm	= 13/16"
w	Web width	=	6.1	mm	
	Area of holes in webs	=	377	mm^2	
A_g	Gross area	=	5752	mm^2	
A_n	Net area	=	5375	mm^2	
A_{ne}	Effective net area	=	4569	mm^2	= 0.85 An
F_u	Tensile strength	=	420	MPa	
T_r	Factored tensile resistance	=	1549	kN	$= 0.85\phi_s A_{ne} F_u$

Section TC2 - Properties

Member Type/Location:	**TC2**
Member Description:	**2-C 10 X 15.3** (cover plate neglected)

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flanges	2	66	11	248.5	1463	14949	363428	121	21579622	21594572
Webs	2	6	232	127.0	2827	12662567	358995	0	0	12662567
Bottom Flanges	2	66	11	5.5	1463	14949	8099	121	21579622	21594572
					5752		730522			55851710

Overall Section Depth:	254	mm	Section Modulus Top (S_x):	4.40E+05	mm^3
Centroid Distance From Bottom of Section:	127	mm	Section Modulus Bottom (S_x) :	4.40E+05	mm^3
Section Moment of Inertia (I_x):	5.59E+07	mm^4	Radius of Gyration (r_x) :	98.5	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange 1	1	66	11	33.0	731	265803	24149	160.0	18727327	18993131
Top Flange 2	1	66	11	353.1	731	265803	258212	160.0	18727327	18993131
Web 1	1	6	232	63.0	1413	4377	89031	130.0	23903509	23907886
Web 2	1	6	232	323.1	1413	4377	456641	130.0	23903509	23907886
Bottom Flange 1	1	66	11	33.0	731	265803	24149	160.0	18727327	18993131
Bottom Flange 2	1	66	11	353.1	731	265803	258212	160.0	18727327	18993131
					5752		1110394			123788294

Clear Gap Between Channels:	254	mm			
Overall Section Width:	386	mm	Section Modulus Top (S_y):	6.41E+05	mm^3
Centroid Distance From Right Side of Section:	193	mm	Section Modulus Bottom (S_y):	6.41E+05	mm^3
Section Moment of Inertia (I_y):	1.24E+08	mm^4	Radius of Gyration (r_y) :	146.7	mm

Section TC2 Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
F_u	Tensile strength	=	420	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	Cl. 10.4.2
b_1	Top flange width	=	66	mm	= 2.6"
t_1	Top flange thickness	=	11	mm	= 0.436"
h	Web height	=	232	mm	
w	Web width	=	6	mm	= 0.24"
b_2	Bottom flange width	=	66	mm	= 2.6"
t_2	Bottom flange thickness	=	11	mm	= 0.436"
b_3	Cover plate width	=	406	mm	= 16"
t_3	Cover plate thickness	=	10	mm	= 0.375"
d_1	Clear gap between channels	=	254	mm	= 10"
L	Unsupported length	=	7160	mm	= 23.5'
Ag	Gross area for compression	=	5752	mm^2	
Iy	Weak axis moment of inertia	=	5.59E+07	mm^4	
Ix	Strong axis moment of inertia	=	1.24E+08	mm^4	
ry	Weak axis radius of gyration	=	98.5	mm	
rx	Strong axis radius of gyration	=	146.7	mm	

SECTION CLASSIFICATION (Cl. 10.9.2)

Flanges in Compression

b	Flange width	=	66	mm	
t	Flange thickness	=	11	mm	
b/t		=	6.0		
F_y	Plate yield strength	=	210	MPa	
	Class 3 limit	=	13.8		= 200 / sqrt(F_y)
	Top flange class	=	Class 3		

Web

h	Web height	=	232	mm	
w	Web width	=	6	mm	
h/w		=	38.0		
F_y	Plate yield strength	=	210	MPa	
	Class 1/2/3 limit	=	46.2		= 670 / sqrt(F_y)
	Web class	=	Class 1		

Cover Plate

h	Plate width between channels	=	254	mm	
w	Cover plate thickness	=	10	mm	
h/w		=	26.7		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	36.2		= 525 / sqrt(F_y)
	Cover plate class	=	Class 1		

Section is Class 3 or less.

Section TC2 Resistance

COMPRESSION MEMBERS (10.9)

Check Slenderness (10.8.1.2)

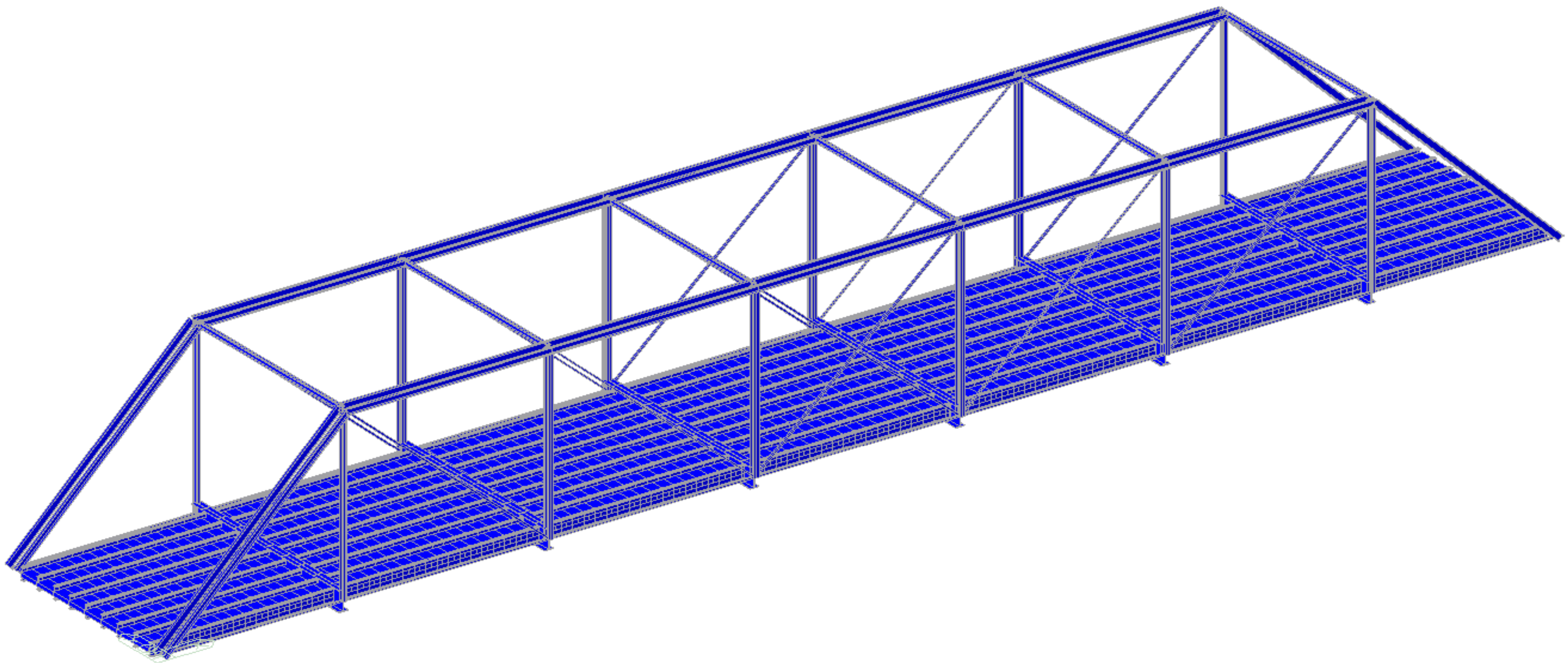
k	Effective length factor	=	1.0		
L	Unsupported length	=	7160	mm	
r	Min. radius of gyration	=	98.5	mm	
	Slenderness ratio kL/r	=	72.7		
	Limit	=	120		
	Acceptable	=	Yes		

Flexural Buckling (10.9.3.1)

ϕ_s	Resistance factor (compression)	=	0.90		Cl. 10.5.7
A	Area of section	=	5752	mm^2	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
L	Unbraced length	=	7160	mm	
n	Coefficient for buckling resistance	=	1.34		
k	Effective length factor	=	1.0		
λ	Slenderness parameter	=	0.75		$= k\,L/r\,(F_y / \pi^2 E_s)$
C_r	Factored compressive resistance	=	819	kN	$= \phi_s\, A\, F_y\, (1+\lambda^{2n})^{-1/n}$

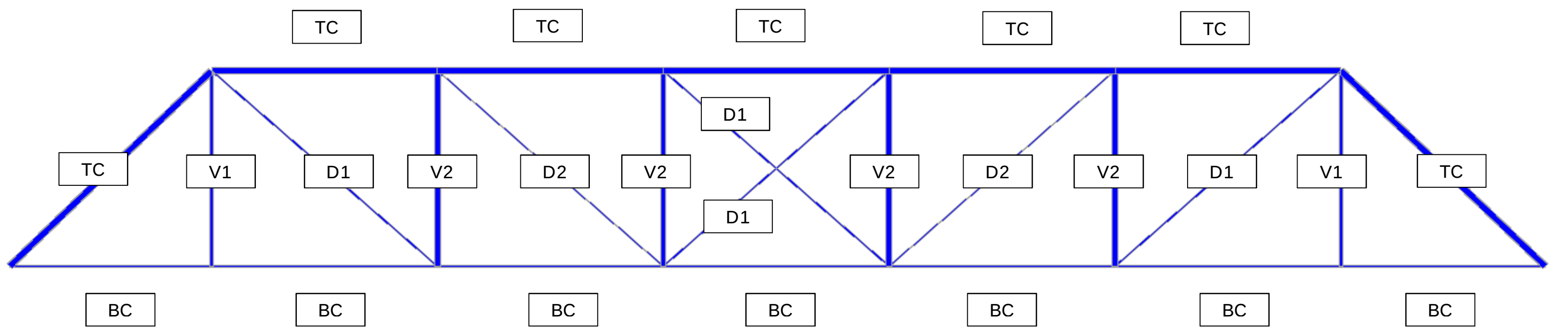
Model Views

3D Model View



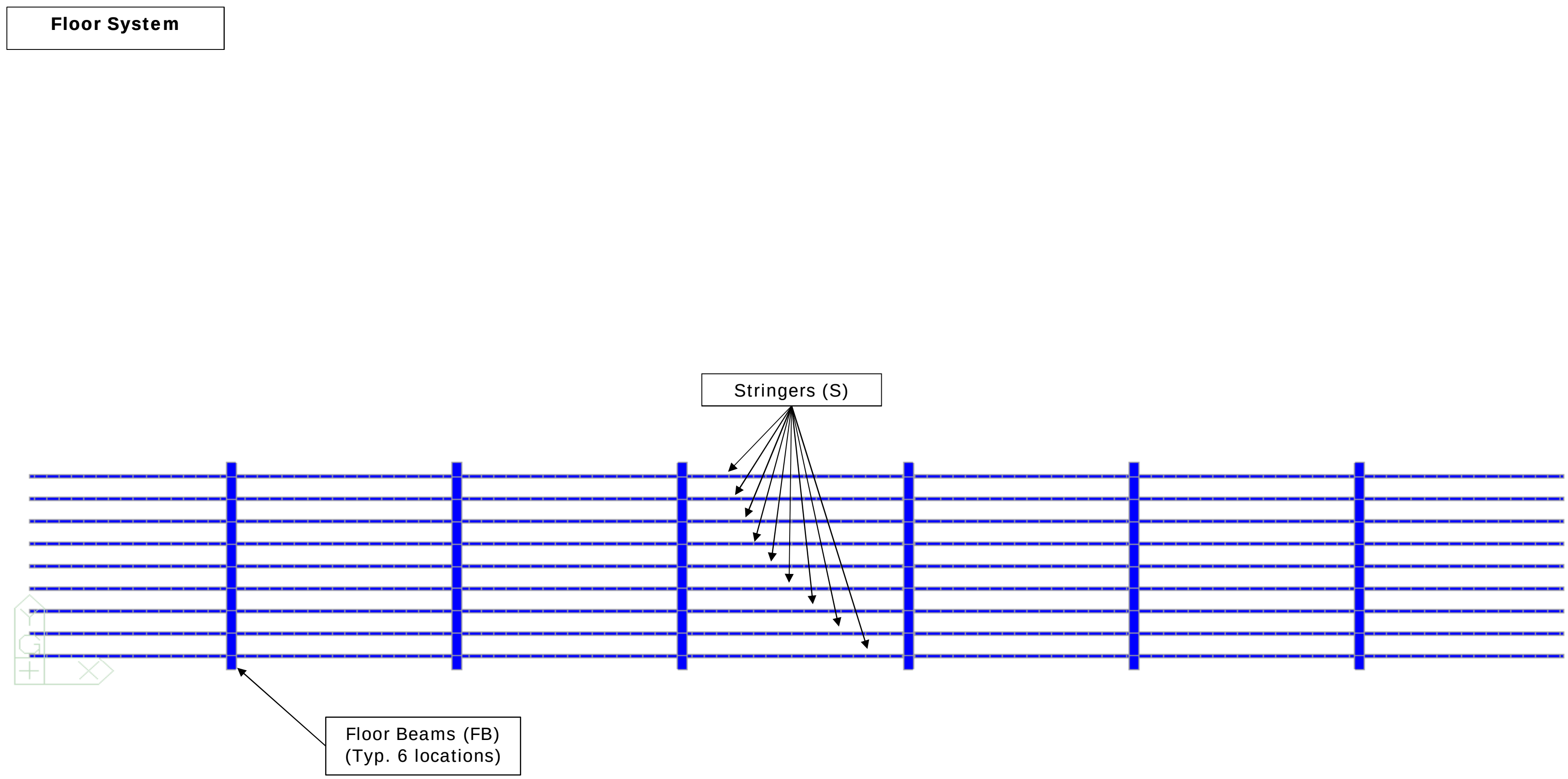
Model Views

Truss Members



PARKS CANADA - CENTRAL ONTARIO
Hamlet Bridge - Fixed Span Evaluation

Model Views



PARKS CANADA BRIDGES IN CENTRAL ONTARIO
Hamlet Bridge Fixed Span

Estimated Span Weight

Description	Component	Section	Material Density kg/m^3	Section Area mm^2	Unit Weight kg/m	No.	Length m	Mass kg	Nominal Mass tonnes	Added %	Mass Added tonnes	Total Mass tonnes	Notes
TRUSSES													
Bottom Chord	Square Bars	2 - 1"x1"	7850	1290	10.1	2	30.9	626	0.6	2%	0.01	0.6	
Top Chord	Channels	C6 x 2	7850	3080	24.2	2	34.7	1677	1.7	2%	0.03	1.7	8.2 lbs/ ft
	Cover Plate	PL 12-1/4" x 5/16"	7850	2470	19.4	2	34.7	1345	1.3	5%	0.07	1.4	
Vertical 1		I 5 x 3	7850	1850	14.5	4	4.4	253	0.3	2%	0.01	0.3	10 lbs/ft
Vertical 2		Unknown section	7850	3500	27.5	8	4.4	958	1.0	2%	0.02	1.0	Field-measured
Diagonals 1 and 5	Square Bars	2 - 1-1/8"	7850	1635	12.8	4	6.3	323	0.3	0%	0.00	0.3	
Diagonals 2 and 4	Square Bars	2 - 7/8"	7850	986	7.7	4	6.3	195	0.2	0%	0.00	0.2	
Diagonal 3	Square Bars	2 - 1"	7850	1290	10.1	4	6.3	255	0.3	0%	0.00	0.3	
											Total trusses mass	**5.8**	**tonnes**
												5770	kg
												56.6	kN
BRACES, PORTAL FRAME ETC.													
Top Braces	Round Bar	1-1/4"	7850	790	6.2	10	6.5	400	0.4	0%	0.00	0.4	
Top Struts		PL 6" x 3/8" + 2L 2-	7850	3665	28.8	6	4.6	789	0.8	2%	0.02	0.8	Assumed sections
Portal Frames etc.		L 2-1/2" x 2-1/2 x 3/8"	7850	1120	8.8	1	25.0	220	0.2	2%	0.00	0.2	Assumed sections
Below-Deck Braces	Round Bar	1-1/4"	7850	790	6.2	14	6.5	564	0.6	0%	0.00	0.6	
											Total braces mass	**2.0**	**tonnes**
												1993	kg
												19.6	kN
FLOOR SYSTEM													
Floor Beams		I 15" x 5-1/2"	7850	8058	63.3	6	4.6	1735	1.7	2%	0.03	1.8	42.9 lbs/ft
												1.8	*tonnes*
												0.3	*tonnes / floor beam*
												2.9	*kN / floor beam*
												0.63	*kN / m / floor beam*
Stringers	Unknown section	I 12" x 3-1/8"	7850	2656	20.8	9	31.0	5817	5.8	2%	0.12	5.9	
												5.9	*tonnes*
												0.7	*tonnes / stringer*
												6.5	*kN / stringer*
												0.21	*kN / m / stringer*
											Total floor system mass	**7.7**	**tonnes**
												7703	kg
												76	kN

PARKS CANADA BRIDGES IN CENTRAL ONTARIO
Hamlet Bridge Fixed Span

Estimated Span Weight

Description	Component	Section	Material Density kg/m³	Section Area mm²	Unit Weight kg/m	No.	Length m	Mass kg	Nominal Mass tonnes	Added %	Mass Added tonnes	Total Mass tonnes	Notes
RAILINGS													
Railing	Railing	2" pipe	7850	693	5.4	6	31.0	1012	1.0	5%	0.05	1.1	
											Total railings mass	**1.1**	**tonnes**
												1062	kg
												10	kN
TIMBER DECK													
Deck		2" x 4"	968	3380	3.3	845	4.3	11893	11.9	0%	0.00	11.9	
Running Boards		2" x 50"	968	63500	61	2	31.0	3813	3.8	0%	0.00	3.8	
Curbs		2-5/8" x 6"	968	10161	10	2	31.0	610	0.6	0%	0.00	0.6	
		4" x 6"	968	15484	15	2	31.0	930	0.9	0%	0.00	0.9	
											Total timber mass	**17.2**	**tonnes**
												17245	kg
												169	kN

TOTAL BRIDGE WEIGHT	**33.8**	**tonnes**
	331.3	*kN*

Notes

- Steel member section areas are without consideration of section loss
- Lengths and sizes are not exact

PARKS CANADA BRIDGES - CENTRAL ONTARIO
Hamlet Bridge - Fixed Span Evaluation

Level 1 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_l (kN)	M_l (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Floor Beams	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	6.7	7.7	14.3	18.2	23	29	216	220	342	191	1.04	0.52
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	1.0	1.8	1.5	3	3	114	53	224	51	1.44	0.67
	S-	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	0.7	1.8	1.3	3	2	114	28	224	43	1.44	1.08
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.0	0.0	0.0	0.0	NA	0	NA	23	NA	24	NA	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords	TC	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.7	70.0	78.3	171	626	780	0.57
Verticals	V2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	15.2	12.5	31	177	186	0.54

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	17.6	19.9	40	177	529	2.04
Bottom Chords	BC	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.7	33.0	37.6	81	317	257	0.33
Verticals	V1	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	9.0	14.2	25	216	382	1.16
Diagonals	D1	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	34.6	38.5	83	372	326	0.40
	D2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	16.8	18.1	40	255	197	0.38

COMBINED AXIAL TENSION AND BENDING - STRINGERS

Member	Fact. Tensile Resistance	Factored Moment Resistance	Effects Due to Dead Loads		Effects Due to Live Loads		F
	Tr	Mr	Tfd	Mfd	Tfl	Mfl	
	kN	kNm	kN	kNm	kN	kNm	
Stringers (S-)	529	43.0	40	2	239	38	0.66

F = (TrMr - Mr Tfd - TrMfd) / (MrTfl + Tr Mfl)

Delcan

Level 2 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_l (kN)	M_l (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Floor Beams	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	6.7	7.7	14.3	18.2	23	29	216	220	342	191	1.04	0.52
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	1.0	1.8	1.5	3	3	114	53	224	51	1.44	0.67
	S-	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	0.7	1.8	1.3	3	2	114	28	224	43	1.44	1.08
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.0	0.0	0.0	0.0	NA	0	NA	23	NA	24	NA	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords	TC	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.7	70.0	78.3	171	582	780	0.62
Verticals	V2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	15.2	12.5	31	170	186	0.56

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	17.6	19.9	40	165	529	2.19
Bottom Chords	BC	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	33.0	37.6	78	283	257	0.42
Verticals	V1	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	9.0	14.2	25	216	382	1.16
Diagonals	D1	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	34.6	38.5	80	341	326	0.51
	D2	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	16.8	18.1	38	245	197	0.46

COMBINED AXIAL TENSION AND BENDING - STRINGERS

Member	Fact. Tensile Resistance	Factored Moment Resistance	Effects Due to Dead Loads		Effects Due to Live Loads		F
	Tr	Mr	Tfd	Mfd	Tfl	Mfl	
	kN	kNm	kN	kNm	kN	kNm	
Stringers (S-)	529	43.0	40	2	223	38	0.67

F = (TrMr - Mr Tfd - TrMfd) / (MrTfl + Tr Mfl)

Level 3 Results

MOMENT AND SHEAR CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects				Factored Dead Load Effects		Unfact. Live Load Effects		Factored Resistances (R_r)		Live Load Capacity Factor (F)	
										D1		D2									
						D1	D2	D3		V (kN)	M (kN.m)	V (kN)	M (kN.m)	V_f (kN)	M_f (kN.m)	V_f (kN)	M_f (kN.m)	V_r (kN)	M_r (kN.m)	V	M
Floor Beams	FB	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	6.7	7.7	14.3	18.2	23	29	216	220	342	191	1.04	0.52
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	1.0	1.8	1.5	3	3	114	53	224	51	1.44	0.67
	S-	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.8	0.7	1.8	1.3	3	2	114	28	224	43	1.44	1.08
Deck	D	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	0.0	0.0	0.0	0.0	NA	0	NA	23	NA	24	NA	0.77

COMPRESSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		C (kN)	C (kN)	C_f (kN)	C (kN)	C_r (kN)	C
Top Chords	TC	S1	E1	INSP3	3.75	1.10	1.20	1.50	1.7	70.0	78.3	171	458	780	0.78
Verticals	V2	S2	E1	INSP3	3.50	1.09	1.18	1.45	1.63	15.2	12.5	31	150	186	0.63

TENSION CHECKS

Member	Section	System Behaviour	Element Behaviour	Inspection Level	Target Reliability Index, β	Dead Load Factor, α_D			Live Load Factor, α_L	Unfactored Dead Load Effects		Factored Dead Load Effects	Unfact. Live Load Effects	Factored Resistances (R_r)	Live Load Capacity Factor (F)
										D1	D2				
						D1	D2	D3		T (kN)	T (kN)	T_f (kN)	T (kN)	T_r (kN)	T
Stringers	S+	S3	E3	INSP3	2.50	1.05	1.10	1.25	1.35	17.6	19.9	40	131	529	2.76
Bottom Chords	BC	S1	E3	INSP3	3.00	1.07	1.14	1.35	1.49	33.0	37.6	78	224	257	0.54
Verticals	V1	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	9.0	14.2	25	216	382	1.16
Diagonals	D1	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	34.6	38.5	80	278	326	0.62
	D2	S2	E3	INSP3	2.75	1.06	1.12	1.30	1.42	16.8	18.1	38	213	197	0.52

COMBINED AXIAL TENSION AND BENDING - STRINGERS

Member	Fact. Tensile Resistance	Factored Moment Resistance	Effects Due to Dead Loads		Effects Due to Live Loads		F
	Tr	Mr	Tfd	Mfd	Tfl	Mfl	
	kN	kNm	kN	kNm	kN	kNm	
Stringers (S-)	529	43.0	40	2	177	38	0.72

F = (TrMr - Mr Tfd - TrMfd) / (MrTfl + Tr Mfl)

Deck Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

b	Section width	=	38	mm	= 1.5"
d	Section height	=	89	mm	= 3.5"
S_x	Elastic section modulus	=	5.02E+04	mm^3	= 1/6 * bh^2

FLEXURAL RESISTANCE (Cl. 9.6.1)

ϕ	Resistance factor (flexure)	=	0.90		Table 9.1
k_d	Load duration factor	=	1.00		Cl. 9.5.3, dead and live loads
k_{ls}	Lateral stability factor	=	1.00		Table 9.5, with d/b <=1
k_m	Load-sharing factor	=	1.40		Cl. 9.5.6
k_{sb}	Size effect factor	=	1.70		Table 9.4
f_{bu}	Bending at extreme fibre	=	8.40		Table 9.12; for SPF 1/2
M_r	Factored moment resistance	=	0.90	kN.m	= $\phi k_d k_{ls} k_m k_{sb} f_{bu} S$
	x (1000/38) for a 1 metre wide section	=	23.8	kNm	

Section S Properties

Member Type/Location:	**Stringer - Section S1**
Member Description:	**I 12" x 3"**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\ d^3/12$	Ay	y_c	$A\ y_c^2$	$I_x + A\ y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	79	4.8	303.0	381	732	115443	150	8623151	8623883
Web	1	6	295	152.5	1888	13691933	287920	0	6	13691940
Bottom Flange	1	79	4.8	2.4	381	732	914	150	8590513	8591244
					2650		404277			30907067

Overall Section Depth:	305	mm	Section Modulus Top (S_x):	2.03E+05	mm^3
Centroid Distance From Bottom of Section:	153	mm	Section Modulus Bottom (S_x) :	2.03E+05	mm^3
Section Moment of Inertia (I_x):	3.09E+07	mm^4	Radius of Gyration (r_x) :	108.0	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\ d^3/12$	Ax	x_c	$A\ x_c^2$	$I_y + A\ x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	79	4.8	39.7	381	200037	15121	0.0	0	200037
Web	1	6	295	39.7	1888	6444	74930	0.0	0	6444
Bottom Flange	1	79	4.8	39.7	381	200037	15121	0.0	0	200037
					2650		105172			406519

Overall Section Width:	79	mm	Section Modulus Top (S_y):	1.02E+04	mm^3
Centroid Distance From Right Side of Section:	40	mm	Section Modulus Bottom (S_y):	1.02E+04	mm^3
Section Moment of Inertia (I_y):	4.07E+05	mm^4	Radius of Gyration (r_y) :	12.4	mm

STRONG AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	79	5	303.0	381	381	151	0	0	57341
Web	1	6	295	152.5	1888	944	74	944	74	139240
Bottom Flange	1	79	5	2.4	381	0	0	381	150	57188
					2650	1325		1325		253769

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	153	mm
Overall Section Depth:	305	mm
Plastic Section Modulus Z_x:	2.54E+05	mm^3

WEAK AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	79.4	4.8	39.7	381	191	20	191	20	7560
Web	1	6.4	295.0	39.7	1888	944	2	944	2	3021
Bottom Flange	1	79.4	4.8	39.7	381	191	20	191	20	7560
					2650	1325		1325		18142

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	40	mm
Overall Section Depth:	79	mm
Plastic Section Modulus Z_y:	1.81E+04	mm^3

Section S Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	
b_1	Top flange width	=	79	mm	
t_1	Top flange thickness	=	4.8	mm	
h	Web height	=	295	mm	
w	Web width	=	6.4	mm	
b_2	Bottom flange width	=	79	mm	
t_2	Bottom flange thickness	=	4.8	mm	
L	Unsupported length	=	2200	mm	In negative bending
Z_x	Plastic modulus	=	2.54E+05	mm^3	
S_x	Elastic section modulus (top)	=	2.03E+05	mm^3	
S_x	Elastic section modulus (bottom)	=	2.03E+05	mm^3	
I_y	Moment of inertia	=	4.07E+05	mm^4	
r_y	Radius of gyration	=	12.4	mm	
r_x	Radius of gyration	=	108.0	mm	
J	Torsional constant	=	3.16E+04	mm^4	Figure C10.2
d_1		=	300	mm	$h + t_1/2 + t_2/2$
C_w	Warping constant	=	8.99E+09	mm^6	Figure C10.2
β_x	Coefficient of monosymmetry	=	0.0		Figure C10.2
A_w	Shear area	=	1949	mm^2	

SECTION CLASSIFICATION (Cl. 10.9.2)

Top Flange in Compression

b	Half flange width	=	40	mm	
t	Flange thickness	=	4.8	mm	
b/t		=	8.3		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Top flange class	=	Class 1		

Web in Flexural Compression

h	Web height	=	295	mm	
w	Web width	=	6.4	mm	
h/w		=	46.1		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	75.9		$= 1100 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Web in Axial Compression

	Class 1 limit	=	46.2		$= 670 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1

Section S Resistance

FACTORED MOMENT RESISTANCE (Cl. 10.10.2)

ϕ_s	Steel resistance factor (bending)	=	0.95		Cl. 10.5.7
F_y	Yield strength	=	210	MPa	

Laterally Supported Members (Class 1 or 2 Sections)

Top flange is fully supported by timber deck anchors for positive bending.

M_p	Plastic moment	=	53.3	kN.m	= Zx Fy
M_r	Factored moment resistance (positive)	=	50.6	kN.m	= ϕs Zx Fy

Laterally Unsupported Members (Class 1 or 2 Sections)

Bottom flange is only supported at floor beams for negative bending.

	0.67 M_p	=	35.7	kN.m	
ω_2	Moment gradient coefficient	=	1.80		Based on DL analysis
B_1	Geometric coefficient	=	0.0		Doubly symmetric
B_2	Geometric coefficient	=	1.5		
M_u	Critical elastic moment	=	57	kN.m	
	Mu > 0.67Mp				
M_r	Factored moment resistance	=	43.0	kN.m	Cl. 10.10.2.3

SHEAR RESISTANCE (Cl. 10.10.5.1)

ϕ_s	Steel resistance factor (shear)	=	0.95		Cl. 10.5.7
A_w	Shear area	=	1949	mm^2	
k_v	Shear buckling coefficient	=	5.34		
h/w		=	46.09		
	First limit	=	80.05		
	Second Limit	=	99.03		
F_{cr}	Shear buckling stress	=	121	MPa	=0.577 Fy
F_t	Tension field component	=	0.0	MPa	
F_s	Ultimate shear stress, $F_{cr} + F_t$	=	121	MPa	
V_r	Factored shear resistance	=	224	kN	= $\phi_s A_w F_s$

COMPRESSION MEMBERS (10.9)

k	Effective length factor	=	0.65		
L	Unsupported length	=	2200	mm	
r	Minimum radius of gyration	=	12.4	mm	
	Slenderness ratio	=	115.5		= kL /r

Flexural Buckling (10.9.3.1)

ϕ_s	Resistance factor (compression)	=	0.9		Cl. 10.5.7
A	Area of section	=	2650	mm^2	
F_y	Yield strength	=	210	MPa	
E_s	Modulus of elasticity	=	200000	MPa	
L	Unbraced length	=	2200	mm	
n	Coefficient for buckling resistance	=	1.34		
λ	Slenderness parameter	=	1.19		= kL/r sqrt ($F_y / \pi^2 E_s$)
C_r	Factored compressive resistance	=	246	kN	= $\phi_s A F_y (1+\lambda^{2n})^{-1/n}$

Section S Resistance

AXIAL TENSILE RESISTANCE (10.8.2)

Gross Section Yielding

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	2650	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	529	kN	$= \phi_s A_g F_y$

Section FB Properties

Member Type/Location:	**Floor Beam - Section FB**
Member Description:	**I 15" x 5-1/2"**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\ d^3/12$	Ay	y_c	$A\ y_c^2$	$I_x + A\ y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	140	14	373.0	1960	32013	731080	182	65131542	65163555
Web	1	8	349	191.0	2792	28339033	533272	0	238	28339271
Bottom Flange	1	140	14	8.0	1960	32013	15680	183	65429128	65461141
					6712		1280032			158963967

Overall Section Depth:	381	mm	Section Modulus Top (S_x):	8.35E+05	mm^3
Centroid Distance From Bottom of Section:	191	mm	Section Modulus Bottom (S_x) :	8.34E+05	mm^3
Section Moment of Inertia (I_x):	1.59E+08	mm^4	Radius of Gyration (r_x) :	153.9	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\ d^3/12$	Ax	x_c	$A\ x_c^2$	$I_y + A\ x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	140	14	70.0	1960	3201333	137200	0.0	0	3201333
Web	1	8	349	70.0	2792	14891	195440	0.0	0	14891
Bottom Flange	1	140	14	70.0	1960	3201333	137200	0.0	0	3201333
					6712		469840			6417557

Overall Section Width:	140	mm	Section Modulus Top (S_y):	9.17E+04	mm^3
Centroid Distance From Right Side of Section:	70	mm	Section Modulus Bottom (S_y):	9.17E+04	mm^3
Section Moment of Inertia (I_y):	6.42E+06	mm^4	Radius of Gyration (r_y) :	30.9	mm

STRONG AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	140	14	373.0	1960	1960	182	0	0	356720
Web	1	8	349	191.0	2792	1396	87	1396	87	243602
Bottom Flange	1	140	14	8.0	1960	0	0	1960	183	358680
					6712	3356		3356		959002

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	191	mm
Overall Section Depth:	381	mm
Plastic Section Modulus Z_x:	9.59E+05	mm^3

WEAK AXIS PLASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Area Above Plastic Centroid	Centroid Distance to Plastic Centroid	Area Below Plastic Centroid	Centroid Distance to Plastic Centroid	Moment of Area Around Plastic Centroid
		b	d	y	A					
		mm	mm	mm	mm^2	mm^2	mm	mm^2	mm	mm^3
Top Flange	1	140.0	14.0	70.0	1960	980	35	980	35	68600
Web	1	8.0	349.0	70.0	2792	1396	2	1396	2	5584
Bottom Flange	1	140.0	14.0	70.0	1960	980	35	980	35	68600
					6712	3356		3356		142784

Iterate centroid so difference is zero

Area Difference:	0.0	mm^2
Plastic Centroid Distance From Bottom of Section:	70	mm
Overall Section Depth:	140	mm
Plastic Section Modulus Z_y:	1.43E+05	mm^3

Section FB Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	
b_1	Top flange width	=	140	mm	
t_1	Top flange thickness	=	14	mm	
h	Web height	=	349	mm	
w	Web width	=	8	mm	
b_2	Bottom flange width	=	140	mm	
t_2	Bottom flange thickness	=	8	mm	
Z_x	Plastic modulus	=	9.59E+05	mm^3	
S_x	Elastic section modulus (top)	=	8.35E+05	mm^3	
S_x	Elastic section modulus (bottom)	=	8.34E+05	mm^3	
I_y	Moment of inertia	=	6.42E+06	mm^4	
J	Torsional constant	=	2.12E+05	mm^4	Figure C10.2
d_1		=	360	mm	$h + t_1/2 + t_2/2$
C_w	Warping constant	=	1.51E+11	mm^6	Figure C10.2
β_x	Coefficient of monosymmetry	=	0.0		Figure C10.2
A_w	Shear area	=	2968	mm^2	

SECTION CLASSIFICATION (Cl 10.9.2)

Top Flange in Compression

b	Half flange width	=	70	mm	
t	Flange thickness	=	14	mm	
b/t		=	5.0		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		= 145 / sqrt (Fy)
	Top flange class	=	Class 1		

Web

h	Web height	=	349	mm	
w	Web width	=	8	mm	
h/w		=	43.6		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	166.5		$= 1100 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1.

Section FB Resistance

FACTORED MOMENT RESISTANCE (Cl. 10.10.2)

ϕ_s	Steel resistance factor (bending)	=	0.95		Cl. 10.5.7
F_y	Yield strength	=	210	MPa	

Laterally Supported Members (Class 1 or 2 Sections)

M_p	Plastic moment	=	201	kN.m	$= Z_x F_y$
M_r	Factored moment resistance	=	191	kN.m	

SHEAR RESISTANCE (Cl. 10.10.5.1)

ϕ_s	Steel resistance factor (shear)	=	0.95		Cl. 10.5.7
A_w	Shear area	=	2968	mm^2	
k_v	Shear buckling coefficient	=	5.34		
h/w		=	43.63		
	First limit	=	80.05		
	Second Limit	=	99.03		
F_{cr}	Shear buckling stress	=	121	MPa	= 0.577 Fy
F_t	Tension field component	=	0	MPa	
F_s	Ultimate shear stress, $F_{cr} + F_t$	=	121	MPa	
V_r	Factored shear resistance	=	342	kN	$= \phi_s A_w F_s$

Section BC Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	
F_u	Tensile strength	=	420	MPa	
Ag	Gross sectional area	=	1290	mm2	2- 1" x 1" square bars

AXIAL TENSILE RESISTANCE (10.8.2)

Gross Section Yielding

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	1290	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	257	kN	$= \phi_s A_g F_y$

Section TC Properties

Member Type/Location: **TC1**
Member Description: **2-C 6" x 2" back to back with 12-3/16" x 5/16" cover plate**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Cover Plate	1	310	8.0	153.0	2480	13227	379440	44	4701331	4714557
Top Flanges	2	50	9.5	148.3	950	7145	140885	39	1433090	1440235
Webs	2	5.0	130	74.5	1300	1830833	96850	35	1588897	3419731
Bottom Flanges	2	50	9.5	4.8	950	7145	4560	105	10406107	10413252
					5680		621735			19987774

Overall Section Depth: 157 mm
Centroid Distance From Bottom of Section: 109 mm
Section Moment of Inertia (I_x): 2.00E+07 mm^4

Section Modulus Top (S_x): 4.20E+05 mm^3
Section Modulus Bottom (S_x) : 1.83E+05 mm^3
Radius of Gyration (r_x) : 59.3 mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Cover Plate	1	310	8.0	155.0	2480	19860667	384400	2.8	19679	19880345
Top Flange 1	1	50	9.5	25.0	475	98958	11875	127.2	7683382	7782340
Top Flange 2	1	50	9.5	275.0	475	98958	130625	122.8	7164896	7263854
Web 1	1	5.0	130	47.5	650	1354	30875	104.7	7123058	7124412
Web 2	1	5.0	130	252.5	650	1354	164125	100.3	6541262	6542617
Bottom Flange 1	1	50	10	25.0	475	98958	11875	127.2	7683382	7782340
Bottom Flange 2	1	50	10	275.0	475	98958	130625	122.8	7164896	7263854
					5680		864400			63639763

Clear space between channels: 200 mm
Overall Section Width: 300 mm
Centroid Distance From Right Side of Section: 152 mm
Section Moment of Inertia (I_y): 6.36E+07 mm^4

Section Modulus Top (S_y): 4.31E+05 mm^3
Section Modulus Bottom (S_y): 4.18E+05 mm^3
Radius of Gyration (r_y) : 105.8 mm

Section TC Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
F_u	Tensile strength	=	420	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	Cl. 10.4.2
b_1	Top flange width	=	50	mm	
t_1	Top flange thickness	=	10	mm	
h	Web height	=	130	mm	
w	Web width	=	5	mm	
b_2	Bottom flange width	=	50	mm	
t_2	Bottom flange thickness	=	10	mm	
b_3	Cover plate width	=	310	mm	
t_3	Cover plate thickness	=	7.9	mm	
d_1	Clear gap between channels	=	200	mm	
L	Unsupported length	=	4550	mm	
A_c	Gross section area	=	5680	mm^2	
r	Min. radius of gyration	=	59.3	mm	

SECTION CLASSIFICATION (Cl .10.9.2)

Flanges in Compression

b	Flange width	=	50	mm	
t	Flange thickness	=	10	mm	
b/t		=	5.3		
F_y	Plate yield strength	=	210	MPa	
	Class 3 limit	=	13.8		= 200 / sqrt(F_y)
	Top flange class	=	Class 3		

Web

h	Web height	=	130	mm	
w	Web width	=	5	mm	
h/w		=	26.0		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	46.2		= 670 / sqrt(F_y)
	Web class	=	Class 1		

Cover Plate (Cl. 10.9.2)

h	Cover plate between channels	=	200	mm	
w	Cover plate thickness	=	8	mm	
h/w		=	25.2		
F_y	Plate yield strength	=	210	MPa	
	Class 3 limit	=	36.2		= 525 / sqrt(F_y)
	Cover plate class	=	Class 1		

Section is Class 3.

Section TC Resistance

COMPRESSION MEMBERS (10.9)

k	Effective length factor	=	1.0		
L	Unsupported length	=	4550	mm	
r	Radius of gyration	=	59.3	mm	
	Slenderness ratio kL/r	=	76.7		
	Limit	=	120		
	Acceptable	=	Yes		

Flexural Buckling (10.9.3.1)

ϕ_s	Resistance factor (compression)	=	0.90		Cl. 10.5.7
A	Area of section	=	5680	mm^2	
F_y	Yield strength	=	210	MPa	
E_s	Modulus of elasticity	=	200000	MPa	
L	Unbraced length	=	4550	mm	
n	Coefficient for buckling resistance	=	1.34		
λ	Slenderness parameter	=	0.79		$= kL/r\ (F_y / \pi^2 E_s)$
C_r	Factored compressive resistance	=	780	kN	$= \phi_s A F_y (1+\lambda^{2n})^{-1/n}$

Section D1 Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	
F_u	Tensile strength	=	420	MPa	
Ag	Gross sectional area	=	1635	mm2	Pair of 1-1/8" square bars

AXIAL TENSILE RESISTANCE (10.8.2)

Gross Section Yielding

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	1635	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	326	kN	$= \phi_s A_g F_y$

Section BC Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	
F_y	Yield strength	=	210	MPa	
F_u	Tensile strength	=	420	MPa	
Ag	Gross sectional area	=	986	mm2	Pair of 7/8" square bars

AXIAL TENSILE RESISTANCE (10.8.2)

Gross Section Yielding

ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	986	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	197	kN	$= \phi_s A_g F_y$

Section V1 Properties

Member Type/Location:	**V1**
Member Description:	**I 5" x 3" x 10#**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	76	8	121	608	3243	73568	59	2080728	2083971
Web	1	6.4	109	62.5	698	690682	43600	0	0	690682
Bottom Flange	1	76	8	4.0	608	3243	2432	59	2080728	2083971
					1914		119600			4858623

Overall Section Depth:	125	mm	Section Modulus Top (S_x):	7.77E+04	mm^3
Centroid Distance From Bottom of Section:	63	mm	Section Modulus Bottom (S_x) :	7.77E+04	mm^3
Section Moment of Inertia (I_x):	4.86E+06	mm^4	Radius of Gyration (r_x) :	50.4	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	76	8	38.0	608	292651	23104	0.0	0	292651
Web	1	6	109	38.0	698	2381	26509	0.0	0	2381
Bottom Flange	1	76	8	38.0	608	292651	23104	0.0	0	292651
					1914		72717			587682

Overall Section Width:	76	mm	Section Modulus Top (S_y):	1.55E+04	mm^3
Centroid Distance From Right Side of Section:	38	mm	Section Modulus Bottom (S_y):	1.55E+04	mm^3
Section Moment of Inertia (I_y):	5.88E+05	mm^4	Radius of Gyration (r_y) :	17.5	mm

Section V1 Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
F_u	Tensile strength	=	420	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	Cl. 10.4.2
b_1	Top flange width	=	76	mm	
t_1	Top flange thickness	=	8	mm	
h	Web height	=	109	mm	
w	Web width	=	6.4	mm	
b_2	Bottom flange width	=	76	mm	
t_2	Bottom flange thickness	=	8	mm	
L	Unsupported length	=	4350	mm	
A	Area	=	1914	mm^2	
r_y	Radius of gyration	=	17.5	mm	
r_x	Radius of gyration	=	50.4	mm	

SECTION CLASSIFICATION (Cl. 10.9.2)

Flanges in Compression

b	Flange width	=	76	mm	
t	Flange thickness	=	8	mm	
b/t		=	9.5		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		$= 145 / \mathrm{sqrt}(F_y)$
	Top flange class	=	Class 1		

Web

h	Web height	=	109	mm	
w	Web width	=	6	mm	
h/w		=	17.0		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	46.2		$= 670 / \mathrm{sqrt}(F_y)$
	Web class	=	Class 1		

Section is Class 1.

Section V1 Resistance

COMPRESSION MEMBERS (10.9)

k	Effective length factor	=	1.0		
L	Unsupported length	=	4350	mm	
r	Radius of gyration	=	17.5	mm	
	Slenderness ratio kL/r	=	248		
	Limit	=	120		
	Acceptable	=	No		But primarily a tension member
	Flexural Buckling (10.9.3.1)				
ϕ_s	Resistance factor (compression)	=	0.90		Cl. 10.5.7
A	Area of section	=	1914	mm^2	
F_y	Yield strength	=	210	MPa	
E_s	Modulus of elasticity	=	200000	MPa	
L	Unbraced length	=	4350	mm	
n	Coefficient for buckling resistance	=	1.34		
λ	Slenderness parameter	=	2.56		$= kL/r\ (F_y / \pi^2 E_s)$
C_r	Factored compressive resistance	=	52.1	kN	$= \phi_s A F_y (1+\lambda^{2n})^{-1/n}$

AXIAL TENSILE RESISTANCE (10.8.2)

	Gross Section Yielding				
ϕ_s	Resistance factor (tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	1914	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	382	kN	$= \phi_s A_g F_y$

Section V2 Properties

Member Type/Location:	**V2**
Member Description:	**I 5" x 5" (roughly)**

STRONG AXIS ELASTIC PROPERTIES

Shape	No.	Width	Depth	Centroid Distance to Bottom of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Bottom of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Bottom of Section	Moment of Inertia Around Section Centroid
		b	d	y	A	$I_x = b\,d^3/12$	Ay	y_c	$A\,y_c^2$	$I_x + A\,y_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	113	10	128.0	1074	8074	137408	62	4060245	4068319
Web	1	12	113	66.5	1356	1442897	90174	0	0	1442897
Bottom Flange	1	113	10	5.0	1074	8074	5368	62	4060245	4068319
					3503		232950			9579535

Overall Section Depth:	133	mm	Section Modulus Top (S_x):	1.44E+05	mm^3
Centroid Distance From Bottom of Section:	67	mm	Section Modulus Bottom (S_x) :	1.44E+05	mm^3
Section Moment of Inertia (I_x):	9.58E+06	mm^4	Radius of Gyration (r_x) :	52.3	mm

WEAK AXIS ELASTIC PROPERTIES

Shape	No.	Depth	Width	Centroid Distance to Right Side of Section	Area	Centroidal Moment of Inertia	First Moment of Area Around Right Side of Section	Centroid Distance to Section Centroid	Second Moment of Area Around Right Side of Section	Moment of Inertia Around Section Centroid
		d	b	x	A	$I_y = b\,d^3/12$	Ax	x_c	$A\,x_c^2$	$I_y + A\,x_c^2$
		mm	mm	mm	mm^2	mm^4	mm^3	mm	mm^4	mm^4
Top Flange	1	113	10	56.5	1074	1142293	60653	0.0	0	1142293
Web	1	12	113	56.5	1356	16272	76614	0.0	0	16272
Bottom Flange	1	113	10	56.5	1074	1142293	60653	0.0	0	1142293
					3503		197920			2300859

Overall Section Width:	113	mm	Section Modulus Top (S_y):	4.07E+04	mm^3
Centroid Distance From Right Side of Section:	57	mm	Section Modulus Bottom (S_y):	4.07E+04	mm^3
Section Moment of Inertia (I_y):	2.30E+06	mm^4	Radius of Gyration (r_y) :	25.6	mm

Section V2 Resistance

REFERENCES

1. CAN/CSA S6-06 Canadian Highway Bridge Design Code (CHBDC)

SECTION PROPERTIES

E_s	Modulus of elasticity	=	200000	MPa	Cl. 10.4.2
F_y	Yield strength	=	210	MPa	Cl. 14.7.4.2
F_u	Tensile strength	=	420	MPa	Cl. 14.7.4.2
G_s	Shear modulus	=	77000	MPa	Cl. 10.4.2
b_1	Top flange width	=	113	mm	
t_1	Top flange thickness	=	10	mm	
h	Web height	=	113	mm	
w	Web width	=	12.0	mm	
b_2	Bottom flange width	=	113	mm	
t_2	Bottom flange thickness	=	10	mm	
L	Unsupported length	=	4350	mm	
A	Area	=	3503	mm^2	
r_y	Radius of gyration	=	25.6	mm	
r_x	Radius of gyration	=	52.3	mm	

SECTION CLASSIFICATION (Cl. 10.9.2)

Flanges

b	Flange width	=	113	mm	
t	Flange thickness	=	10	mm	
b/t		=	11.9		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	10.0		= 145 / sqrt(F_y)
	Top flange class	=	Class 2		

Web

h	Web height	=	113	mm	
w	Web width	=	12	mm	
h/w		=	9.4		
F_y	Plate yield strength	=	210	MPa	
	Class 1 limit	=	46.2		= 670 / sqrt(F_y)
	Web class	=	Class 1		

Section is Class 2

Section V2 Resistance

COMPRESSION MEMBERS (10.9)

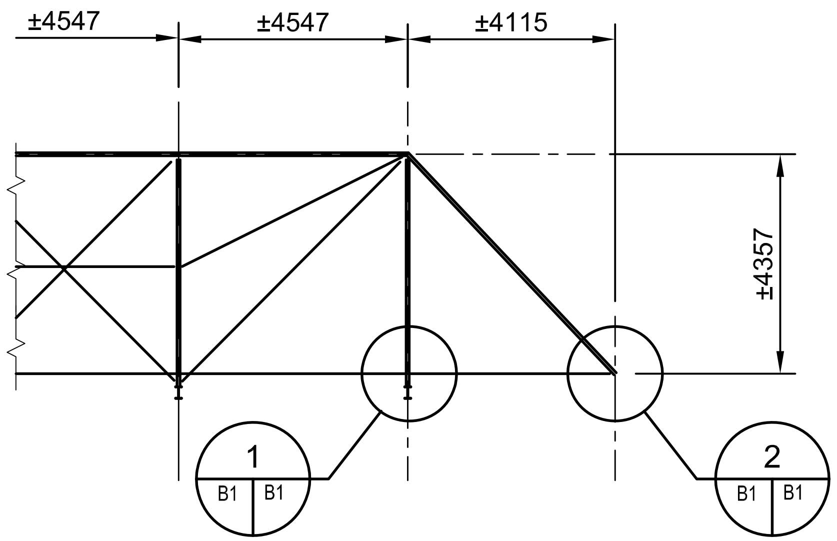
k	Effective length factor	=	1.0		
L	Unsupported length	=	4350	mm	
r	Radius of gyration	=	25.6	mm	
	Slenderness ratio kL/r	=	170		
	Limit	=	120		
	Acceptable	=	No		
	Flexural Buckling (10.9.3.1)				
ϕ_s	Resistance factor (compression)	=	0.9		Cl. 10.5.7
A	Area of section	=	3503	mm^2	
F_y	Yield strength	=	210	MPa	
E_s	Modulus of elasticity	=	200000	MPa	
L	Unbraced length	=	4350	mm	
n	Coefficient for buckling resistance	=	1.34		
λ	Slenderness parameter	=	1.75		$= kL/r\,(F_y / \pi^2 E_s)$
C_r	Factored compressive resistance	=	185.9	kN	$= \phi_s A F_y (1+\lambda^{2n})^{-1/n}$

AXIAL TENSILE RESISTANCE (10.8.2)

	Gross Section Yielding				
ϕ_s	Resistance factor (Tension)	=	0.95		Cl. 10.5.7
A_g	Gross area	=	3503	mm^2	
F_y	Yield strength	=	210	MPa	
T_r	Factored tensile resistance	=	699	kN	$= \phi_s A_g F_y$

APPENDIX L

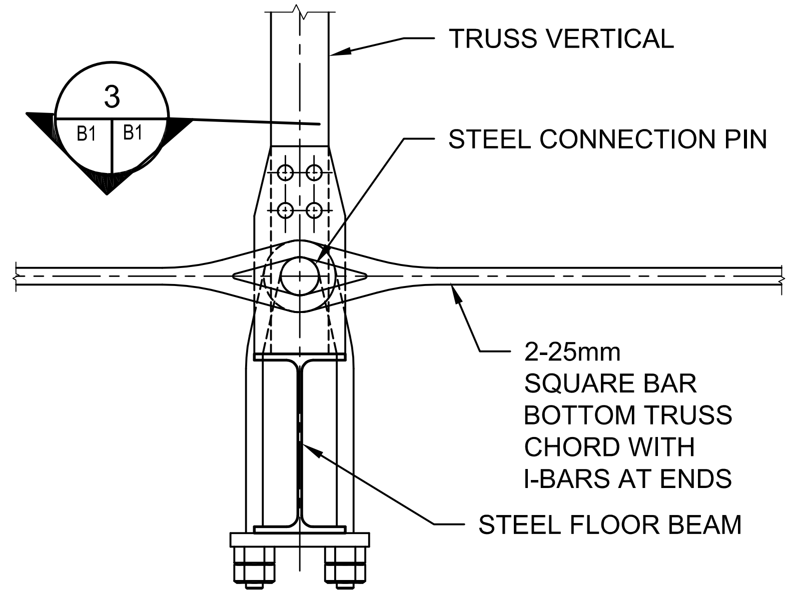
FIXED BRIDGE BOTTOM CHORD TEMPORARY REPAIR



PARTIAL SOUTH ELEVATION

1:150

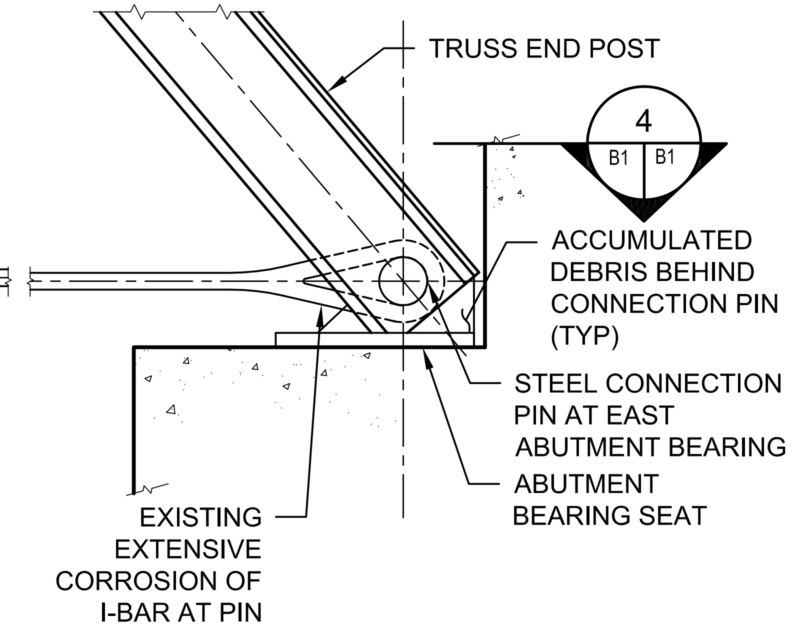
(NORTH ELEVATION SIMILAR)



1 B1 B1 DETAIL

1:10

(EXISTING CONDITION)



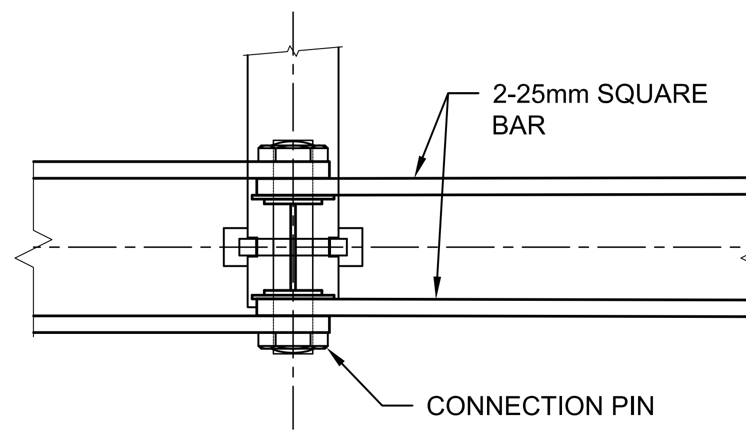
2 B1 B1 DETAIL

1:10

(EXISTING CONDITION)

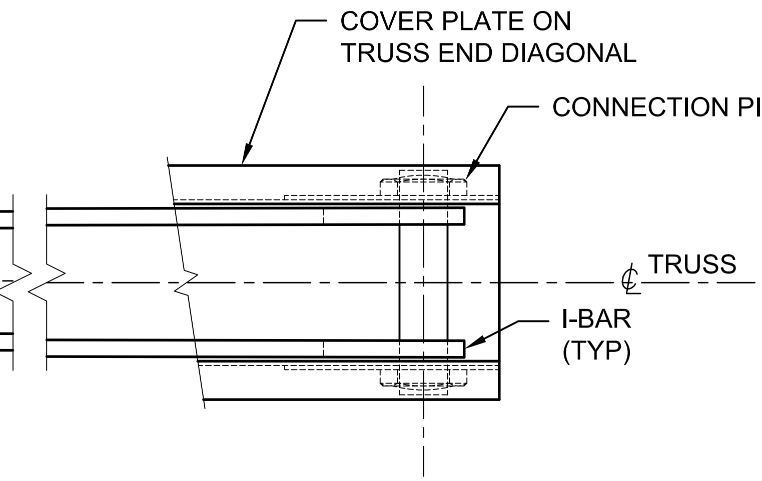
PHOTO AT EAST ABUTMENT

(SOUTH BEARING SHOWN, NORTH BEARING SIMILAR)



3 B1 B1 PLAN DETAIL

1:10



4 B1 B1 PLAN DETAIL

1:10

GENERAL NOTES:

1. ALL DIMENSIONS IN MILLIMETERS UNLESS NOTED OTHERWISE.
2. THE CONTRACTOR IS SOLELY RESPONSIBLE FOR FIELD MEASUREMENT OF THE EXISTING STRUCTURE AND GOOD FIT OF ALL NEW COMPONENTS.
3. THE CONTRACTOR SHALL NOTIFY PARKS CANADA OF ANY DISCREPANCY BETWEEN THE EXISTING STRUCTURE AND THESE DRAWINGS BEFORE PROCEEDING WITH THE WORK.
4. THE PROPOSED REPAIRS ARE REQUIRED AT EACH TRUSS BOTTOM CHORD AT THE EAST END OF THE FIXED SPAN (AT THE EAST ABUTMENT). THE DOWNSTREAM DIRECTION IS TAKEN AS NORTH.
5. THE BRIDGE SHALL REMAIN CLOSED UNTIL THESE REPAIRS ARE COMPLETED AND INSPECTED BY PARKS CANADA AND/OR DELCAN.
6. "APPROVED EQUIVALENT" REFERS TO AN ALTERNATE PRODUCT APPROVED FOR USE BY PARKS CANADA.

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Canada

Delcan

Contractor to verify all dimensions & conditions on site and immediately notify the engineer of all discrepancies.

revisions	description	date
0	FOR CONSTRUCTION	OCT. 12/11

A detail no. / no. du detail
B location drawing no. / sur dessin no.
C drawing no. / dessin no.

project / projet

PARKS CANADA
TRENT-SEVERN WATERWAY

HAMLET BRIDGE 57
FIXED SPAN

drawing / dessin

TEMPORARY BRIDGE REPAIR
EXISTING CONDITION

Designed By / Conçu par	BM	
Date	2011/10/12	(yyyy/mm/dd)
Drawn By / Dessiné par	RD	
Date	2011/10/12	(yyyy/mm/dd)
Reviewed By / Examiné par	PM	
Date	2011/10/12	(yyyy/mm/dd)
Approved By / Approuvé par	DM	
Date	2011/10/12	(yyyy/mm/dd)

Tender / Soumission

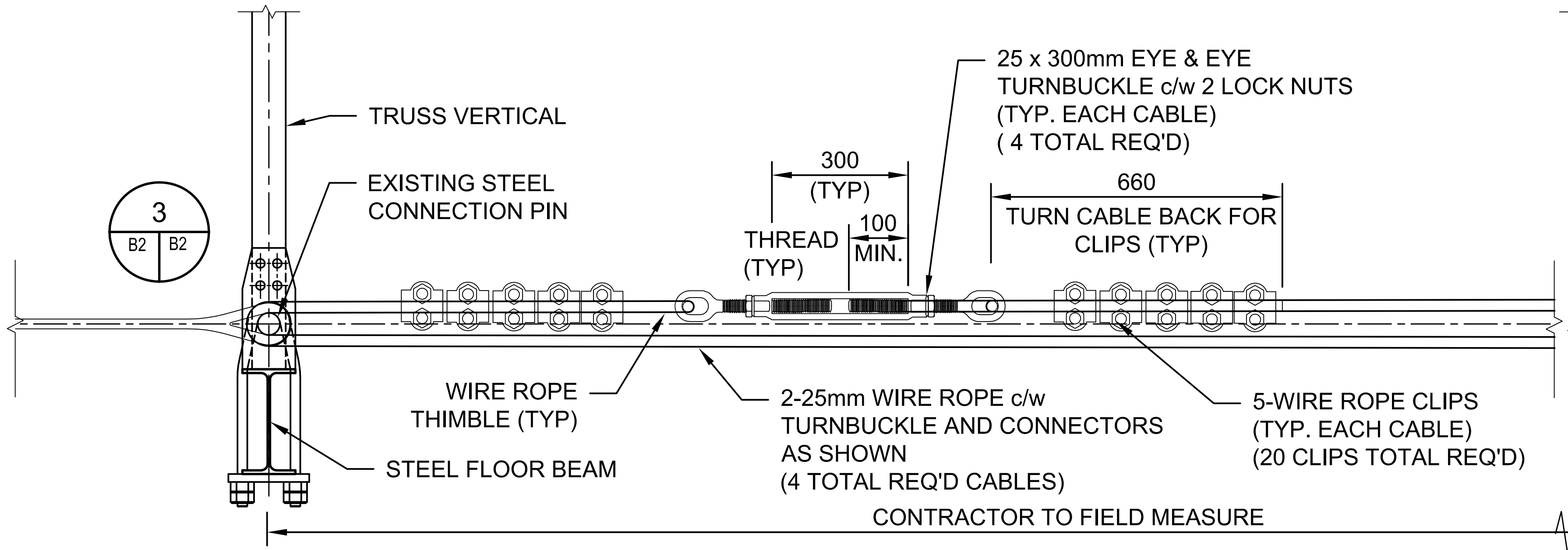
Project Manager / Administrateur de projets

Project no. / No. du projet

Drawing no. / No. du dessin

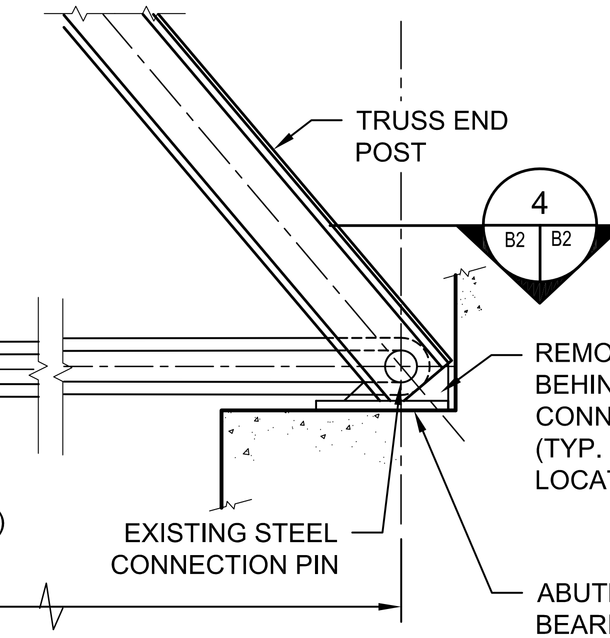
B1

PWGSC 11X17 (431x279)

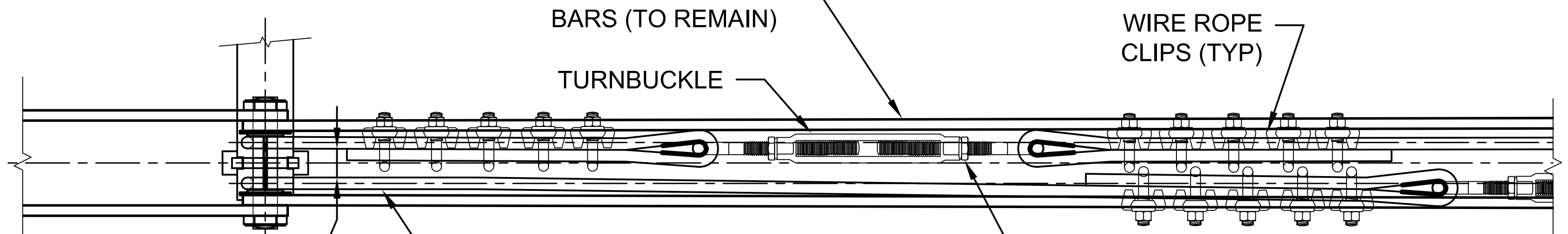


1 B1 B2 DETAIL 1:15 (NEW CONNECTION)

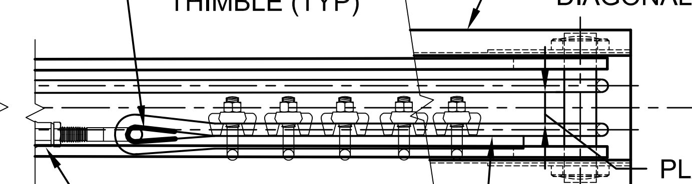
NOTE:
EXISTING TRUSS BOTTOM CHORD NOT SHOWN FOR CLARITY



2 B1 B2 DETAIL 1:15 (NEW CONNECTION)



3 B2 B2 PLAN DETAIL 1:15



4 B2 B2 PLAN DETAIL 1:15

NOTES:

1. WIRE ROPE (CABLE) TO BE 25 mm 6 x 41 (IWRC) GALVANIZED WIRE ROPE, GRADE 110/120 ACCORDING TO CSA G4 OR EQUIVALENT, WITH MINIMUM BREAKING LOAD OF 355kN (40 tons).
2. WIRE ROPE THIMBLES TO BE GALVANIZED G-411 BY CROSBY OR APPROVED EQUIVALENT.
3. WIRE ROPE CLIPS TO BE GALVANIZED G-450 CLIPS BY CROSBY, OR APPROVED EQUIVALENT. NUTS TO BE TORQUED TO 225 ft-lbs MINIMUM.
4. TURNBUCKLES TO BE EYE AND EYE TYPE, GALVANIZED, HG-226 BY CROSBY OR APPROVED EQUIVALENT. TURNBUCKLES TO BE TENSIONED IN ACCORDANCE WITH DIRECTIONS UNDER "REPAIR SEQUENCE", AND SECURED WITH 2 GALVANIZED LOCK NUTS.
5. INSTALL WIRE ROPE CLIPS ORIENTED AS SHOWN, WITH U-BOLTS AROUND SHORT SIDE AND ALL CLIPS ORIENTED THE SAME WAY.
6. CLIPS, TURNBUCKLES AND THIMBLES MAY BE ROTATED FROM ORIENTATION SHOWN, BUT MUST HAVE SAME ORIENTATION RELATIVE TO EACH OTHER.

REPAIR SEQUENCE:

1. CLEAN DEBRIS FROM BEHIND CONNECTION PINS AT EAST ABUTMENT TO MAKE ROOM FOR THE CABLES.
2. INSTALL THE TWO REPAIR CABLES AT THE SOUTH TRUSS EACH WITH A TURNBUCKLE, 2 THIMBLES AND 10 CLIPS. PULL THE CABLES AS TIGHT AS POSSIBLE BY HAND AND TIGHTEN TURNBUCKLES TO TAKE UP ANY SLACK. PLACE CABLES AS FAR APART HORIZONTALLY AS POSSIBLE.
3. TIGHTEN THE NUTS ON THE CLIPS TO 225 ft-lbs USING A TORQUE WRENCH.
4. TIGHTEN THE TURNBUCKLES BY 10mm AND TIGHTEN LOCK NUTS.
5. REPEAT STEPS 2 THROUGH 4 AT THE NORTH TRUSS BOTTOM CHORD.

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Canada

Delcan

Contractor to verify all dimensions & conditions on site and immediately notify the engineer of all discrepancies.

revisions	description	date
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project / projet
PARKS CANADA
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HAMLET BRIDGE 57
FIXED SPAN

drawing / dessin
TEMPORARY BRIDGE REPAIR
PROPOSED REPAIRS

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Tender / Soumission		
Project Manager / Administrateur de projets		
Project no. / No. du projet		
Drawing no. / No. du dessin	B2	