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EB144-150508/A	002
Client Reference No. - N° de référence du client	**Date**
EB144-15-0508	2014-09-17

GETS Reference No. - N° de référence de SEAG
PW-$PWA-115-5118

File No. - N° de dossier	**CCC No./N° CCC - FMS No./N° VME**
PWA-4-72016 (115)	

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La modification no 001 à la demande de soumissions est apportée pour les raisons suivantes:

référence: 3.1.1 Licenses et permis, certification ou autorisation

Supprimer ce qui suit:

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3.1.1 Licenses et permis, certification ou autorisation

Le proposant doit être un(une) [insérer la discipline], [accrédité(e), ou doit pouvoir être accrédité(e), certifié(e) ou autorisé(e)] pour fournir les services professionnels requis, dans toute la mesure prescrite par les lois provinciales ou territoriales en vigueur dans la(le) [province] [territoire] de(du) [insérer le nom de la province ou du territoire].

L'exigence de licences et de certificats s'applique aussi aux membres clés du promoteur de projet, incluant les sous-traitants et spécialistes tel qu'indiqué dans la section 3.1.2 ci-dessous.

référence: 3.1.2 Identification des membres de l'équipe de l'expert-conseil

Supprimer ce qui suit:

Insérer:

3.1.2 Identification des membres de l'équipe de l'expert-conseil

Les membres de l'équipe de l'expert-conseil à identifier sont les suivants :
Note au rédacteur : Le gestionnaire de projet est tenu d'identifier les disciplines ou les spécialités choisies pour le proposant et pour les principaux sous-experts- conseils. Le contenu et la longueur de la liste varieront en fonction de chaque projet.
Proposant (expert-conseil principal) - [discipline] [spécialité]
Principaux sous-experts-conseils / spécialistes - [discipline] [spécialité]

Si le soumissionnaire propose de fournir des services pluridisciplinaires qui pourraient normalement être fournis par un sous-expert-conseil, il doit l'indiquer ici.

Renseignements requis - nom de l'entreprise et des personnes clés à affecter à la réalisation du projet. En ce qui concerne l'expert-conseil principal, indiquer les accréditations, certifications ou autorisations existantes et/ou les moyens qu'il entend prendre pour respecter les exigences en matière de licences et de permis de la province ou du territoire où le projet sera réalisé. Dans le cas d'une coentreprise, indiquer la forme juridique existante ou proposée de cette dernière (se reporter à l'article IG9 intitulé « Limite quant au nombre de propositions » de la clause R1410T Instructions générales aux proposants).

L'ingénieur civil de structures et l'ingénieur en mécanique doivent être du personnel aîné, avec un minimum de 10 ans d'expérience dans le domaine de la conception de pont mobile en acier.

Un exemple d'un formulaire acceptable (typique) pour la présentation des renseignements relatifs à l'identification des membres de l'équipe, est présenté à l'annexe A.

référence: 3.2.7 Principes/approche/méthodologie de conception

Supprimer ce qui suit:

Insérer:

3.2.7 Principes/approche/méthodologie de conception

Le proposant aurait avantage à préciser certains aspects du projet considérés comme défi principal, qu'illustreront sa philosophie, son approche et sa méthodologie de conception. Le proposant a ici l'occasion de décrire la philosophie de conception globale de l'équipe ainsi que l'approche qu'elle entend utiliser pour résoudre les questions relatives à la conception et, en particulier, de fournir des explications détaillées sur des aspects uniques du projet actuel.

Préparer un plan qui décrit l'approche de conception, les matériaux, les méthodes de construction, ainsi que les autres techniques et méthodologies misent en oeuvre pour assurer le remplacement du pont dans les délais prévus. Ce plan sera évalué pour déterminer s'il propose une approche de conception et de construction qui minimise la période de fermeture au trafic automobiles et piétonniers sur le canal St. Peter's ainsi que des possibilités d'accélérer les travaux de construction. Description des enjeux importants et de la démarche retenue par l'équipe pour les surmonter. Nota : la circulation pour traverser le canal doit être maintenue pendant la construction. Des fermetures de courte durée (20-30 minutes) seront permises seulement entre 22 h et 5 h et sur approbation préalable de Parcs Canada et du ministère des Transports de la Nouvelle-Écosse.

***Toutes les autres conditions demeurent inchangées.**

Questions et réponses

Q1 : *À la page 12 (de 61) de la DP, il y a une liste des capacités requise pour l'équipe de consultation. Nous remarquons que le terme « hydraulique » est indiqué. S'agit-il de la conception des systèmes mécaniques/hydrauliques ou de l'étude hydrologique du canal? Nous remarquons qu'il n'y a aucune exigence relative à la réalisation d'une étude hydrologique indiquée ailleurs dans la DP.*

R1 : La référence concerne la conception des systèmes mécaniques/ hydrauliques.

Q2. Dans le document *R1410T GI9 (2013-04-25) Limite quant au nombre de propositions, à la remarque 4 il est indiqué qu'« ...un proposant ne doit pas inclure dans sa soumission un autre proposant comme membre de son équipe d'expert-conseil que ce soit à titre de sous-expert-conseil ou expert-conseil spécialisé ». Peut-on renoncer à cette exigence pour ce qui est de la DP actuelle?*

R2 : ***Non.***

Q3. *La section RS6 de la DP intitulée Administration de la construction et du contrat contient une liste d'articles à prix fixe. Nous remarquons que le niveau d'effort requis pour exécuter la portée des travaux dépend fortement de la qualité de l'entrepreneur choisi et de la durée totale de la construction, qui pourrait être hors du contrôle du consultant. L'effort requis pour réaliser certaines activités, comme l'examen des dessins d'atelier et les inspections sur place sont en bout de ligne un produit de la qualité des travaux originaux examinés. Nous remarquons que la DP présente un calendrier d'exécution des travaux plutôt serré avec les documents d'appel d'offres prêts à être soumis d'ici le 31 janvier 2016 et la construction à terminer d'ici le 15 mai 2017 (en supposant 15,5 mois pour l'appel d'offres et l'adjudication du contrat. De plus, la SA2 indique qu'il doit y avoir supervision de la mi-septembre 2016 à la mi-juin 2017 (ce qui donne une période de construction de 9 mois). En tenant compte du fait que la majeure partie de l'effort fourni pour ces travaux est directement lié à la durée totale de la construction et à la qualité des travaux réalisés par l'entrepreneur, comment TPSGC prévoit traiter les services supplémentaires potentiels requis par le consultant?*

R3 : Les changements apportés aux travaux seront traités conformément aux conditions de la demande de proposition.

Q4. Dans la *DP 8, à la section 8.2, il est indiqué qu'« un pont découvert est préférable s'il n'ajoute pas au coût de la structure. » Le SR2, section 2.2, remarque 2d indique que les conceptions doivent « doivent comporter un tablier pour véhicules léger et plein sur toute sa longueur.» Est-ce une préférence de TPSGC ou de Parc Canada d'avoir un pont ouvert ou fermé?*

R4 : Un pont ouvert est préférable car il n'ajoute pas au coût de la structure. Les ponts pleins et léger seront également examinés dans les options de conception.

Q5 : *J'aimerais vous demander d'avoir accès à certains rapports mentionnés dans la demande de propositions sur le remplacement du pont tournant du canal de St. Peters. La section 4.1 de la DP mentionne les documents existants suivants qui sont disponibles pour consultation par tous les proposants :*

1. Highway Bridge Evaluations for Parks Canada in Atlantic Provinces - St. Peter's Canal Historic Site Bridge Evaluation Report, par la McCormick Rankin Corporation (volumes 1 et 2), février 2007
2. Electrical, Mechanical and Structural Steel Inspection of St. Peter's Canal Bridge, par la McCormick Rankin Corporation, janvier 2010

R5 : Les documents sont joints à la présente modification.

TOUTES LES AUTRES CONDITIONS DEMEURENT INCHANGÉES.

PUBLIC WORKS AND
GOVERNMENT SERVICES
CANADA

Electrical, Mechanical and Structural Steel Inspection of the St. Peters Canal Bridge

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May 28/2010

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May 28/2010

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MRC

A member of MMM GROUP

McCORMICK RANKIN CORPORATION
January 2010

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Appendices

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Appendix B Photographs
Appendix C Mechanical Inspection and Electrical
Appendix D Cost Estimates

File: W:\7k\7780 St. Peters Bridge Insp. and Rehab\7780.300 Structural\304 - Reports\7780 St.Peters Canal Bridge (Draft Report).doc

EXECUTIVE SUMMARY

McCormick Rankin Corporation (MRC) was retained by Public Works and Government Services Canada (PWGSC) to undertake an inspection of the St. Peter's Canal Bridge for Parks Canada based on our 2006 Inspection.

The main focus of the inspection was to carry out the detailed electrical/mechanical inspection of the bridge, detailed inspection of the structural steel components and non destructive testing of the fatigue sensitive details.

Visual Inspection of the structural steel was carried out along with hammer sounding and probing to assess the current conditions of the various structural steel components.

The electrical and mechanical inspection of the bridge was completed on November 26, 2009, by a subconsultant, Byrne Engineering. The structural inspection was completed on December 2 and 3, 2009 by McCormick Rankin Corporation Staff.

The access for the structural steel inspection was from scaffolding on a barge for the span over the canal, and from scaffolding and ladders for the tail span. The counterweights were removed and reinstated to permit the full inspection of the steel truss members.

The structural steel is in fair to poor condition. The bottom chords also exhibited medium to severe corrosion particularly at the panel points. Perforations of various sizes along with many areas of section loss were noted at the gusset plates, lacing bars and the truss members. Many rivets and bolts were noted with severe section loss from corrosion. In some instances the rivet heads or nuts have been completely consumed by corrosion.

Coating breakdown was noted at approximately 10%-20% of the structural steel. Localized areas of severe coating breakdown were noted on the bottom chords of the trusses, the floor beams and stringers. The previous cleaning and coating of structural steel was completed in 1991. Based on the observed condition, the coating is no longer providing protection to many of the critical steel components and should be recoated as soon as possible.

MRC recommends that the rehabilitation strategy should include steel repairs and reinforcement at locations with perforations and sections losses and the replacement of rivets and bolts having severe section loss from corrosion. The coating is in very poor condition and MRC would recommend that cleaning and coating of structural steel should be carried out within one (1) to three (3) years as noted above.

The estimated cost of the repair to the structural steel is $ 461,000.00. The cost to clean and coat the structural steel is estimated to be $ 678,000.00.

Recommendations for the electrical and mechanical rehabilitation work for St. Peter's Canal Bridge have been provided by Byrne Engineering. A recommended time range to perform the rehabilitation work has also been provided by Byrne Engineering.

The estimated cost of electrical and mechanical rehabilitation work that is recommended for consideration in the one to five year horizon is $ 834,000.00 which includes the wedge system replacement, control panel relocation and bogey wheels replacement.

The electrical and mechanical work that is recommended for rehabilitation at this time includes the new hydraulic fluid, new hydraulic support hardware, new sight glass indicator, new junction and terminal boxes, new flexible cable and cabling at the west traffic gates. The estimated cost for the components recommended for rehabilitation at this time is $ 91,000.00.

1. INTRODUCTION

McCormick Rankin Corporation was retained by PWGSC to undertake a detailed comprehensive inspection of the St. Peter's Canal Bridge for Parks Canada.

Access was provided by Super Port Marine using scaffold on a barge for the span over the canal. Access to the reminder of the bridge was provided by scaffold and ladders for "hands on" inspection.

The Mechanical and Electrical inspections were completed by Apurva Patel and Theresa Dobbing of Byrne Engineering on November 26, 2009.

MRC completed the field inspection on December 2 and 3, 2009 under the supervision of Doug Dixon, P. Eng. The field work was completed by Ubaid Khan, P. Eng. (lead) with assistance from Giovanni Italiano.

Non destructive testing (magnetic particle testing) of some of the fatigue prone details was completed by Andrew Milne of Acuren.

This report presents the following:

a) Observations of the existing conditions and summary of significant findings for the structural, mechanical and electrical inspection;

b) Photographs of the observed condition of the bridge;

c) Rehabilitation recommendations; and

d) Cost estimates to repair the identified conditions.

2. BACKGROUND

The St. Peter's Canal Bridge is a steel pony truss bob-tailed swing bridge. The deck is an open steel grating except at the tail end (counterweight span), where a concrete slab exists. The structural steel floor system consists of steel deck grating welded to support beams which are supported on longitudinal stringers and transverse floor beams.

The truss is fabricated from built up steel components consisting of angles and plate. Solid web plates and lacing bars are used to complete the built up members. Past reinforcement has been undertaken by welding of plates to many of the members.

St. Peter's Canal Bridge is located 20 meters east of Deney's Street on Highway 4 in St. Peter's, Nova Scotia. Photographs 1 and 2 in Appendix B show the general views of the bridge.

The bridge was constructed circa 1936 and rehabilitated in 1982, 1991 and 1997/1998. There are no known original bridge drawings.

In 1991, the structural steel was cleaned and coated.

The 1991 rehabilitation included the installation of new decking and steel beam guide rails on the bridge. The decking consisted of concrete slab at the counterweight span (between nodes 2W and 3W), and open steel grating over the remaining sections.

The 1997 rehabilitation included the following: patch repairs to the abutments and pier; removed, inspected, and refurbished the pintle bearing assembly; approach improvements by installing steel beam guide rails, approach parapet walls, and traffic loop detectors; and reinforced various truss diagonals and top chords.

A sidewalk was added to the south side of the bridge. Construction year of the sidewalk is unknown as there are no known rehabilitation drawings. At the time when the new sidewalk was added, concrete counterweights were added to the bottom chord of the north truss to provide transverse balance to the new sidewalk.

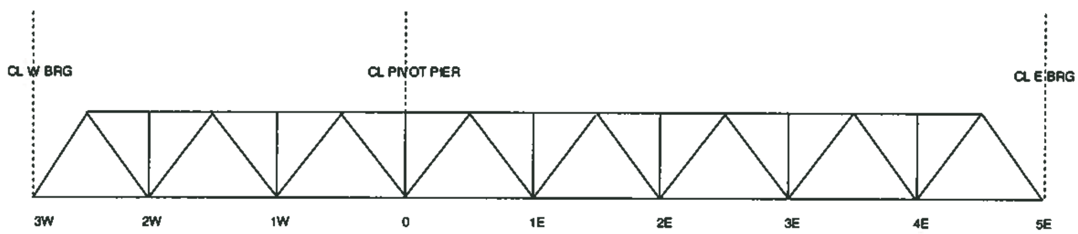
The current inspection was predominantly a visual inspection of the structural (augmented with limited non-destructive testing) mechanical and electrical components to assess the condition of the bridge and identify areas of recommended remedial work. The bridge was observed in operation during the mechanical and electrical inspection.

In general the identification of functional or operational deficiencies of the bridge has not formed part of our investigation. Deficiencies in horizontal and vertical alignment of the approaches or the superelevation or sight distance of the roadway have not been assessed.

3. STRUCTURAL STEEL INSPECTION

For the purpose of this report, the bridge is considered to be oriented in an east-west direction (east is towards Sydney). The location of the bridge is shown on the Key Plan in Appendix A. The stringers (S) are numbered from S1 to S8 from north to south, except between nodes 2W and 3W (counter weight span), where there are only 6 stringers (numbered S1 to S6 from north to south). The node numbers for the purpose of reference are shown in the following Figure 1 and are adopted from previous reports and drawings.

Figure 1- Node Identification



For the purpose of discussion, the structure is divided into a north and south truss, deck system, bracing (horizontal) and ancillary steel.

3.1 North Truss

The north truss components above deck were noted to be in good to fair condition with localized areas of light, medium and severe corrosion. Lacing bars at the top chord between nodes 0 to 2E were noted to have section loss as shown in Photographs 3 and 4. Selective rivets and bolts at a number of locations had extensive section loss having undergone severe corrosion as shown in Photographs 5 and 6. A number of bolts are too short and have insufficient thread engaged by the nuts as show in Photograph 7. The bolts appear to be a mixture of high and low strength bolts throughout the structure.

In order to carry out the inspection of the bottom chord along the north truss, the concrete counter weights attached to the bottom chord were lowered (as shown in Photograph 8). This enabled MRC to inspect the condition of the bottom chord.

Below deck truss components, particularly the bottom chord, exhibited medium to severe corrosion as shown in Photograph 9 and 10. Numerous perforations were noted at the web and flanges of the bottom chord. Perforation at the web of the bottom chord at panel point 3W is show in Photographs 11 and 12. Significant section loss can be noted throughout many lower chord members.

Lacing bars along the bottom of the chord between panel points 3E and 5E had section loss and perforations as shown in Photograph 13. MRC observed that the lacing bars at the connection points also had severe corrosion with pack rust.

The riveted and bolted connections for the lower chord of the north truss were noted to be in good to fair condition. Rivets heads were noted to have section loss. Bolts at many locations along the truss were corroded and had section loss within the nuts.

The diagonal and vertical members of the north truss were in good to fair condition with localized perforations and section loss noted at several locations. A typical perforation and section loss at a vertical member at panel point E4 is shown in the Photograph 14.

The condition of the gusset plates at the panel point locations along the bottom chord varied from good to poor. Perforations and section loss were noted. Typical conditions are shown in Photographs 15, 16 and 17. Using the Ultrasonic thickness gauge, MRC recorded the section loss at gusset plate locations where we visually observed the gusset plates had undergone section loss. We noted that the section loss at gusset plates at W2 (Interior Plate), E1 (Interior Plate) and E3 (Interior Plate) varied from 20% - 50%.

Numerous reinforcements have previously been completed to the north truss components by welding as shown in Photographs 18 and 19. Dynamically loaded structures such as bridges are subjected to variations in forces in the members. Movable bridges are even more unique with stress reversals in many members at some point during the movement of the bridge. Welded repair/reinforcement is generally not desirable in the tension zone, due to fatigue.

Truss members are typically reinforced due to inadequate tension or compression capacity for plain bolted or riveted members. The act of reinforcing the members by welding, generally creates weld fatigue details with stress range limits set by codes which are far less than the original member (without reinforcement). The reinforcement must be designed with this reduced capacity in mind.

In addition, reinforcement welded to a member undergoes compression in the reinforcement material as the weld metal shrinks (cools). This creates additional residual compressive forces in an already existing compression member. As it is usually impossible to quantify the magnitude of this residual compressive stress, the effectiveness of the welded reinforcement material is generally questionable at best.

Inspection of the welded reinforcements added to the structure is discussed in detail in Section 3.6.

Generally, the coating for the north truss was observed to be in fair to poor condition. The above deck coating was in fair condition with coating breakdown over 5% to 10 % of the area. The below deck coating was in poor to very poor condition with coating break down over more than 20% of the area at the connections, panel point locations and along the top and underside of the bottom chord. The concrete counter weights along the bottom chord at the north truss have also contributed to the poor condition of the coating at the bottom chord. The moisture from the bridge deck containing salts and chemicals as well as debris becomes trapped between the concrete counter weight and underside of the web of the bottom chord creating an environment which accelerates coating breakdown and promotes the corrosion process.

3.2 South Truss

The south truss components above deck were in good to fair condition with localized areas of light, medium and severe corrosion. Lacing bars at the top chord between nodes 0 to 1W, 2W to 3W and 0 to 1E were noted to have section loss as shown in Photograph 20. Numerous rivets and bolts had section loss and had undergone severe corrosion.

The bottom chord exhibited medium to severe corrosion as shown in Photograph 21. Numerous perforations were noted at the web and flanges of the bottom chord.

Lacing bars along the bottom chord between panel points 3E and 5E had section loss and perforations as shown in Photograph 22. We noted that the lacing bars at the connection points had severe corrosion and pack rust.

The riveted and bolted connections for the lower chord of the truss were noted to be in good to fair condition. Some rivets heads were noted to have section loss. A number of bolts along the truss were corroded with significant section loss.

The diagonal and vertical members of the south truss were in good to fair condition with localized perforations and section loss noted at some locations. Typical perforation and section loss at the vertical member at panel point E4 is shown in the Photograph 23.

The condition of the gusset plates at the panel point locations along the bottom chord varied from good to poor. Perforations and section loss were noted. Typical conditions are shown in Photographs 24 and 25. Measurements of the section loss at gusset plates at W2 (Interior Plate), 0 (Interior Plate) and E3 (Interior and Exterior Plates) varied from 20% to 50%.

As with the north truss a number of reinforcements have been completed to the south truss components by welding. This is shown in Photographs 26 and 27.

The coating for the south truss was observed to be in fair to poor condition. The above deck coating was in fair condition with coating breakdown over 5% to 10 % of the area. The below deck coating was in poor to very poor condition with coating break down over 20% of the area at the connections, panel point locations, along the top and underside of the bottom chord.

3.3 Deck System

The decking system consists of a galvanized open steel grating except at the concrete counterweight span between nodes 2W and 3W. The floor system consists of decking beams, stringers and floor beams.

The galvanized steel deck grating was in good condition with light corrosion.

The transverse decking beams and the longitudinal stringers were in good condition with light to medium corrosion at the connections and along the bottom flange. Perforations and section loss at S8 between nodes 2W and 1W were noted as shown in Photograph 28. Typical condition of stringers and decking beams are shown in Photographs 29 and 30.

The transverse floor beams were generally noted to be in good to fair condition. It was noted that reinforcement has been welded at some floor beam locations along the bottom flanges (as shown in Photograph 31) and also on the web of the floor beams close to the connection angles (where the floor beam connects to the truss section). The end sections of the floor beams at panel point locations had medium to severe corrosion at the web and bottom flanges. The angles connecting the floor beams to the main truss were noted to be in poor condition at select locations with medium to severe corrosion along with section loss as shown in Photograph 32.

The web of the end floor beams (facing the ballast walls) had no coating protection as shown in Photograph 33. We believe the web of the end floor beam was probably not coated in 1991 (the time of the last cleaning and coating). Using the ultra sonic thickness gauge, the web thickness of the end floor beams was measured. Although unprotected, it was noted that the end floor beam had only minimal section loss.

Generally the deck system was noted to be in good to fair condition with light to medium corrosion. Coating breakdown at the deck system was noted to be approximately 10 % to 20 % of the area.

3.4 Wind Bracing

The horizontal/wind bracing was noted to be generally in fair condition as shown in Photographs 34 and 35. Typical medium to severe corrosion of the bracing was noted at the panel point connections. Coating breakdown was estimated at over 20% of the area of the bracing. Perforation and section loss was noted at the bracing connecting at panel point 5E at the south truss.

3.5 Ancillary Structural Steel

The sidewalk structural steel at the east abutment was noted to be in poor condition with medium to severe corrosion as shown in Photographs 36 and 37. The transverse and the longitudinal steel channels supporting the timber sidewalk have perforations and section loss. At one transverse support location it was noted that a hollow structural section (HSS) was welded to the existing steel anchorage as shown in Photograph 38 and 39. The HSS is supported by a column on the existing ground.

3.6 Non–Destructive Testing

Non-destructive (Magnetic Particle) testing was carried out after the visual inspection of the welded reinforcements at the structural steel components.

Under the supervision of MRC, the welded reinforcements at the top and bottom chords, diagonal members, floor beams and connection angles were tested using the magnetic particle testing (MPT) method. The focus of the non-destructive testing (NDT) was on the possible damaged areas, and fatigue prone welds. Test coverage of the MPT of the welds was approximately 15% to 20% of all of the visually inspected welds.

MPT of the welds was completed by a certified welding inspector from Acuren Group Inc. The welds were inspected and/or tested in accordance with CSA Standard W59-03. MPT of the welds was carried out by utilizing the visible wet method with a prepared bath and white contrast. Following surface preparation, white contrast paint was applied to the bare steel. A prepared bath of metal fillings was sprayed onto the contrast paint and a magnetic field was induced through the area that was tested using a Parker Contour Electromagnetic yoke (A.C). In the presence of a crack in the weld, the dark fillings align themselves with the defect on the white surface, rendering it more visible to the naked eyes.

The magnetic particle testing of the welded reinforcements for the south truss top chord between panel points 2W to 3W and 1E to 3E did not indicate any cracks in the welds. Similarly, the welded reinforcements at diagonal members between panel points 2E and 3E at the south truss top chord did not indicate any cracks in the welds when tested for magnetic particle testing. Photographs 40, 41 and 42 show the magnetic particle testing of the welded reinforcements at the south truss top chord and diagonal members.

The magnetic particle testing of the welded reinforcements for below deck structural steel components did not show any cracks in welds except at one location (south truss bottom chord at panel point 5E) where a crack in the weld was noted. This is shown in Photograph 43 and 44.

General photographs of NDT of the welded reinforcements at below deck structural steel components are shown in Photographs 45 to 50.

4. MECHANICAL INSPECTION

MRC retained a subconsultant, Byrne Engineering to complete the mechanical inspection of the St. Peter's Canal Bridge. The detailed mechanical inspection report provided by Byrne Engineering is attached in Appendix C. The summary of significant findings of the mechanical inspection is as follows.

- The end wedges located at the east end were functional but the components were noted to be corroded with some mechanical damage to the wedge components.

- The main drive machinery consisting of two hydraulic cylinders mounted to the centre pier with the rod yoke attached to the bridge structure was in good condition.

- The centre bearing was noted to be in good condition. The bearing housing retains oil and no leaks were noted. The oil overflow line draining on the west side of the pier was noted to be not contained and could present an environmental issue.

- The hydraulic power unit located in the machinery house was noted to be in good working condition. It was noted that there was no fluid drip tray installed underneath the power unit, as is current practice.

- The gear boxes at the east and west abutment traffic gates were noted to be noisy during operation.

- The bogey wheels that stabilize the bridge during the swing were noted to be showing signs of wear. The bogey wheels have been shimmed over time and the shims were observed to be corroding. Concerns were raised with pack rush which could form between the shims.

- The steel work at both the end stop locations (east abutment and center pier) was noted to be corroded but functional. The rubber however was noted to be cracked.

- The two tail wheels at the west abutment, the wheel housings and the receivers were noted to be corroded and worn. The wheels had been shimmed and shims were observed to be corroding.

- At the time of the inspection, the bridge was noted to be well balanced.

The recommendations and the cost estimate for the rehabilitation work for the mechanical components of the bridge has been discussed in detail in the report (attached in Appendix C) prepared by Byrne Engineering and in Section 6.

5. ELECTRICAL INSPECTION

Byrne Engineering also completed the electrical inspection of the St. Peter's Canal Bridge. The detailed electrical inspection report provided by Byrne Engineering is attached in Appendix C. The summary of significant findings of the electrical inspection is as follows.

- The main electrical distribution panel in the machinery house was noted to be in good condition.

- Field junction boxes and terminal boxes were noted to be in very poor condition and in need of replacement.

- The operators control panel located on the bridge faces the traffic side and it was noted by the operator that it would be preferable to relocate the control panel to a safer location.

- Some of the electrical cabling was noted to be in fair condition (exiting the machinery house), where as the flexible cable at the junction/terminal boxes was noted to be in very poor condition.

The recommendations and the cost estimate for the rehabilitation work for the electrical components of the bridge has been discussed in detail in the report (attached in Appendix C) prepared by Byrne Engineering and in Section 6.

6. DISCUSSION AND RECOMMENDATIONS

Overall we would rate the bridge to be in fair condition.

The above deck structural steel is in good condition. During the above deck inspection, MRC noted severe corrosion and section loss at numerous riveted and bolted connections. Some bolts are short and had insufficient thread engaged to the nut. Lacing bars at the top chord had section loss. The coating for the above deck structural steel components was observed to be in fair condition with coating breakdown over 5% to 10% of the area.

The condition of the below deck structural steel was noted to be ranging from good to poor. Welded reinforcements have been previously completed at various locations along bottom chords, panel points, diagonal members and floor beams. Numerous riveted and bolted connections had severe corrosion and section loss. Section loss and perforations at few gusset plates and along bottom chords at various locations was noted. The lacing bars along the bottom chords had section loss, perforations and severe corrosion at the connections. The coating for the below deck structural steel components was observed to be in poor to very condition with coating breakdown over 10% to 20 % of the area.

Based on the above discussions regarding the structural steel components of the bridge, MRC would recommend following structural steel rehabilitation work.

- Remove and replace approximately 300 rivets/bolts that have undergone severe corrosion and section loss.

- Remove and replace approximately 100 lacing bars that have severe corrosion/section loss and/or perforations.

- Provide additional reinforcement at six (6) panel point locations where gusset plates have undergone section loss and have perforations.

- Reinforce structural components (top chord, bottom chord, floor beams etc.) at approximately 20 locations that have section loss and perforations.

- The existing cleaning and coating of structural steel was completed in 1991.Based on the observed condition, the coating is no longer providing protection to many of the critical steel components. We would recommend that the structural steel should be cleaned and coated as soon as possible.

- During the time of cleaning and coating of structural steel, further steel inspection is recommended to identify any additional areas of steel where repairs become evident while abrasive blast cleaning of the steel.

- As discussed in Section 3.6 the transverse and the longitudinal channels supporting the sidewalk are in poor condition with section loss and perforations. We would recommend that the channels be removed and replaced.

The estimated cost of the structural steel repairs/rehabilitation (not including mechanical and electrical works) work including a 20% contingency is approximately $461,000.00. The detail break down of the cost estimate for the structural steel rehabilitation can be found in Appendix D.

The estimated cost of cleaning and coating of structural steel including a 20% contingency is $678,000.00. The detailed breakdown of the cost estimate for the cleaning and coating of structural steel can be found in Appendix D.

The recommendations and cost estimates for the mechanical and electrical rehabilitation work have been discussed in detail in Byrne's Engineering report in Appendix C.

In Byrne's Engineering report, the mechanical and electrical components that are recommended to be replaced at this time are the hydraulic fluid, hydraulic line support hardware, new sight glass indicator, new junction and terminal boxes, new flexible cable and cabling at the west traffic gate. The above items that have been identified to be replaced now would have a cost estimate of approximately $91,000 that includes a 20% contingency.

The estimated cost for the remaining mechanical and electrical rehabilitation work as noted in Byrne's Engineering report is $834,000.00 that includes a 20% contingency. This includes the wedge system replacement, control panel relocation and bogey wheel replacement. The time frame recommended for rehabilitation of the above electrical and mechanical components ranges from 1-5 years.

The replacement of the control panel noted by Byrne is not based on "function" but rather on a poor "location" as discussed with one of the operators. Future mechanical and electrical inspections can be used to monitor the condition of the wedge system, centre pier bogey wheels and tail wheels for the proper timing of necessary intervention.

In this manner, if Parks Canada wishes, they can defer the repair of mechanical/electrical items that have not been identified as being an immediate need. This would utilize the remaining useful life of the electrical and mechanical components and would allow Parks Canada to plan and budget the work over the next few years. Reliability may be reduced as the repairs are deferred.

APPENDIX A

Key Plan

ST. PETERS CANAL SWING BRIDGE
CAPE BRETON ISLAND

NOVA SCOTIA

St. Peter's Canal Bridge located 20 m east of Deney's St. on Hwy 4 in St. Peter's.

KEY PLAN
N.T.S

APPENDIX B

Photographs

Photograph 1 – North Elevation.

Photograph 2 – West approach looking east.

Photograph 3 – Lacing bars along top chord at north truss. Note: Section loss in lacing bars.

Photograph 4 – Lacing bars along top chord at north truss. Note: Section loss in lacing bars.

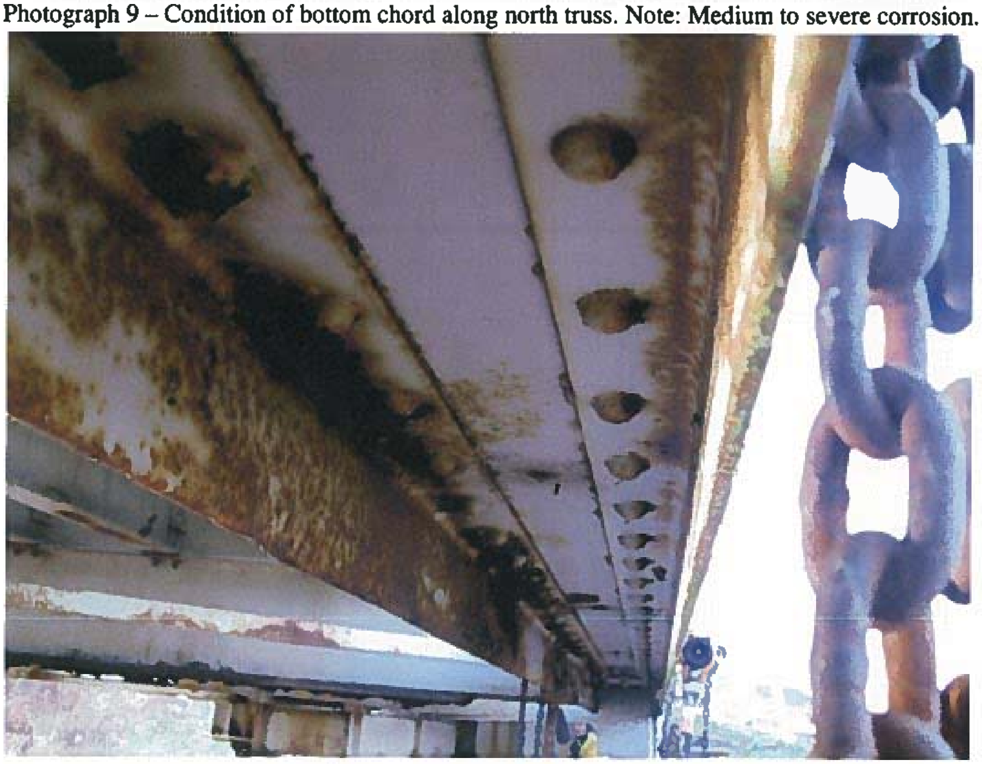
Photograph 5 – Condition of rivets at top chord (node 2W) on north truss. Note: Section loss and corrosion.

Photograph 6 – Condition of bolts at top chord (node 1W) on north truss. Note: Section loss and corrosion.

Photograph 7 – Insufficient bolt thread (typical).

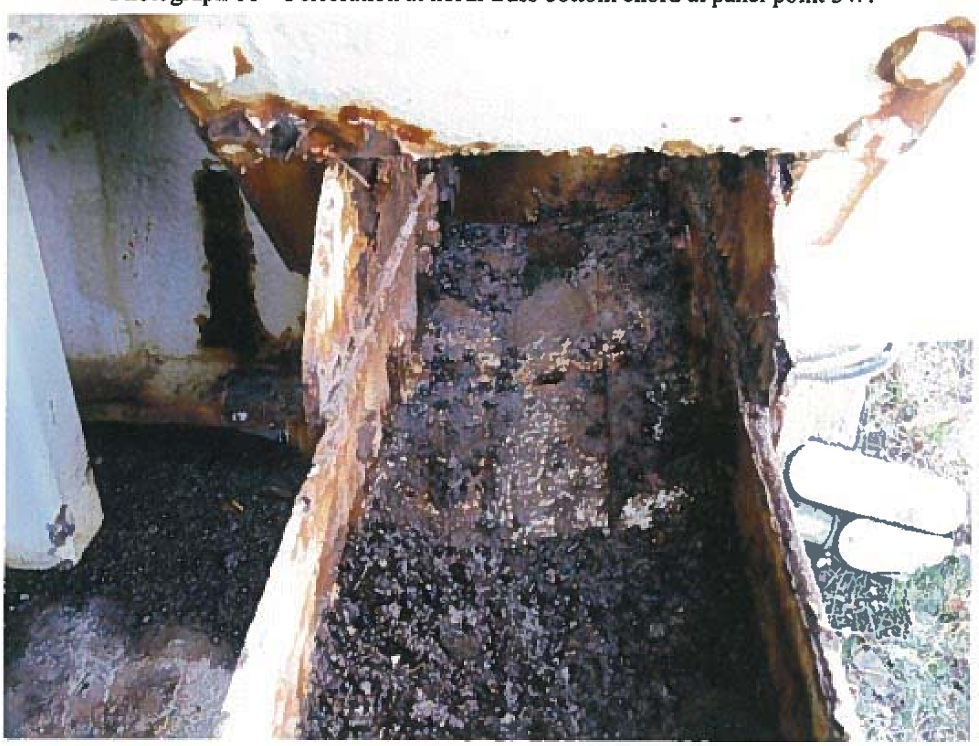
Photograph 8 – Lowering of concrete counter weights along north truss (typ.).

Photograph 9 – Condition of bottom chord along north truss. Note: Medium to severe corrosion.



Photograph 10 - Condition of bottom chord along north truss. Note: Medium to severe corrosion.

Photograph 11 – Perforation at north truss bottom chord at panel point 3W.



Photograph 12 – Perforation at north truss bottom chord at panel point 3W.

Photograph 13 – Perforation / section loss and corrosion at lacing bars at north truss bottom chord.

Photograph 14 – Section loss and perforation of the vertical member at panel point location along north truss bottom chord.

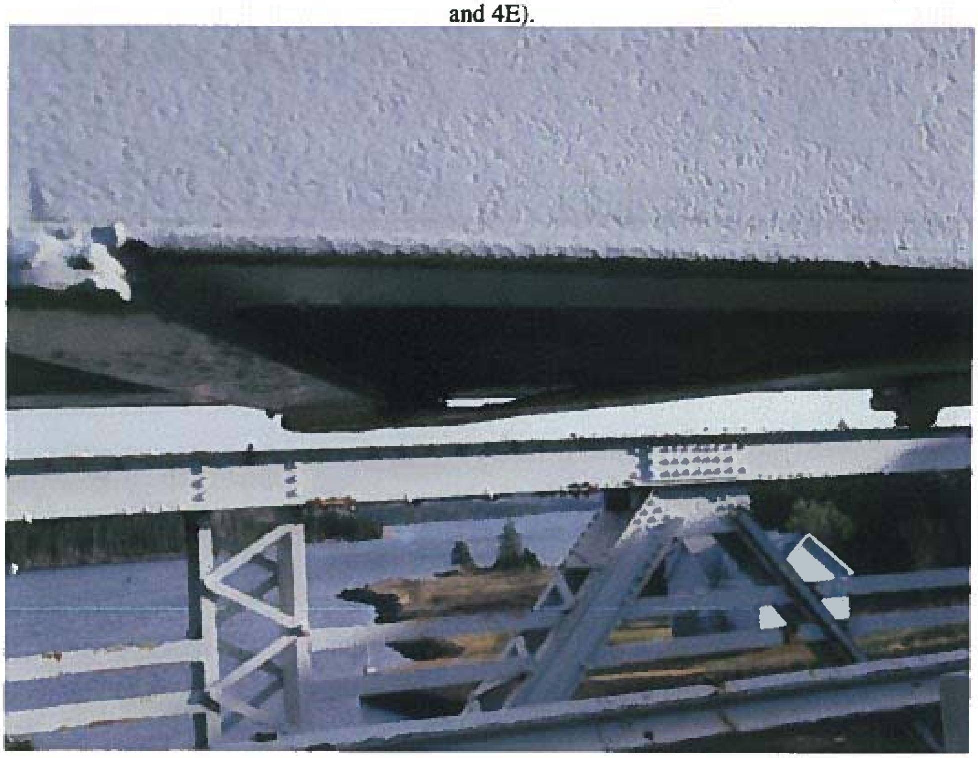
Photograph 15 – Section loss and perforation at gusset plate location at north truss panel point 2W.

Photograph 16 - Section loss and perforation at gusset plate location at north truss panel point 2W

Photograph 17 – Section loss and perforation at gusset plate (panel point 0) at north truss.

Photograph 18 – Welded reinforcement along north truss bottom chord and at panel point locations.

Photograph 19 – Welded reinforcements along north truss bottom chord (between panel points 3E and 4E).

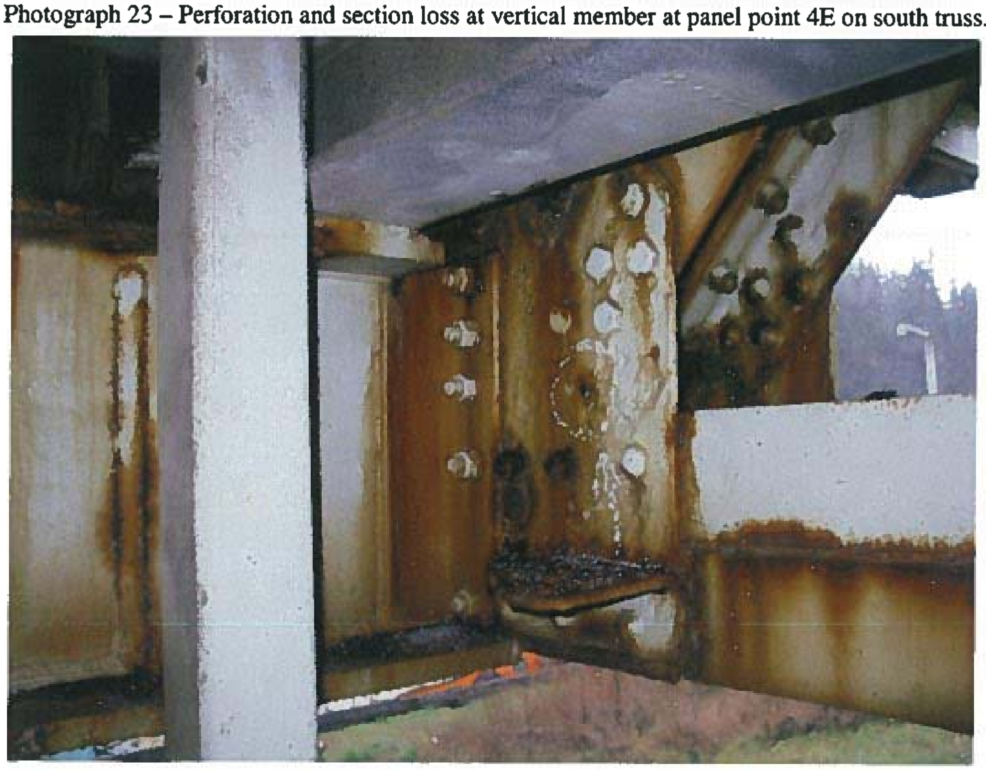


Photograph 20 – Lacing bars along top chord at south truss. Note: Section loss at lacing bars.

Photograph 21 – Condition of bottom chord along south truss. Note: Medium to severe corrosion.

Photograph 22 – Condition of lacing bars along south truss bottom chord between panel points 3E and 5E.

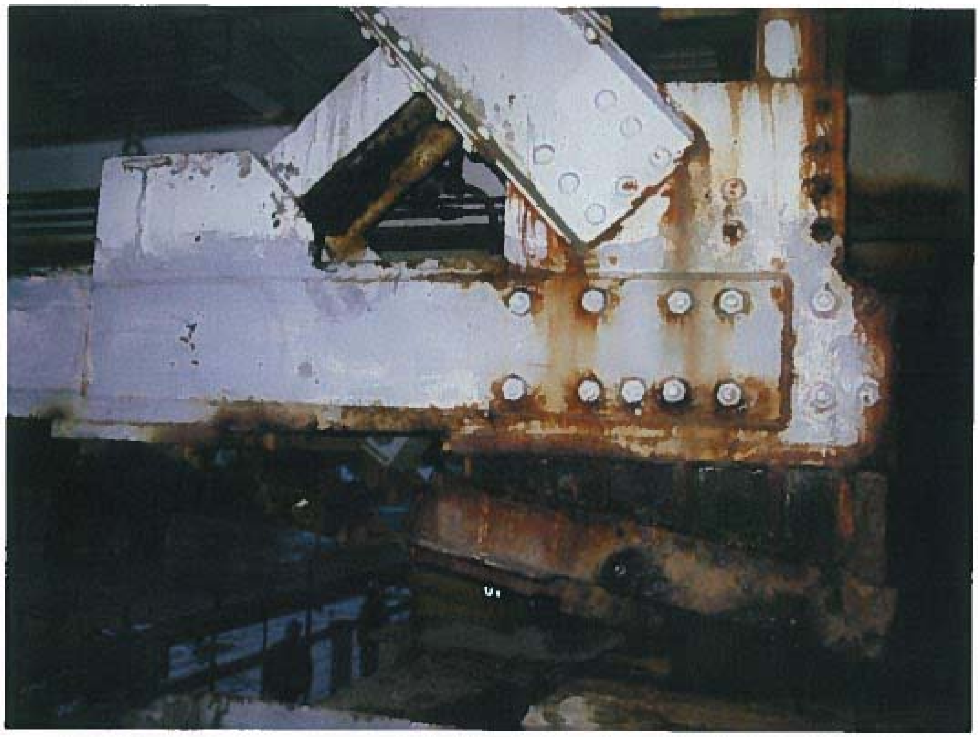
Photograph 23 – Perforation and section loss at vertical member at panel point 4E on south truss.



Photograph 24 – Perforation / section loss at gusset plate panel point 2W south truss bottom chord.

Photograph 25 – Perforation / section loss at gusset plate panel point 0 south truss bottom chord.

Photograph 26 – Welded reinforcement along south truss (between panel points 4E and 5E).



Photograph 27 – Welded reinforcement along south truss at the nose.

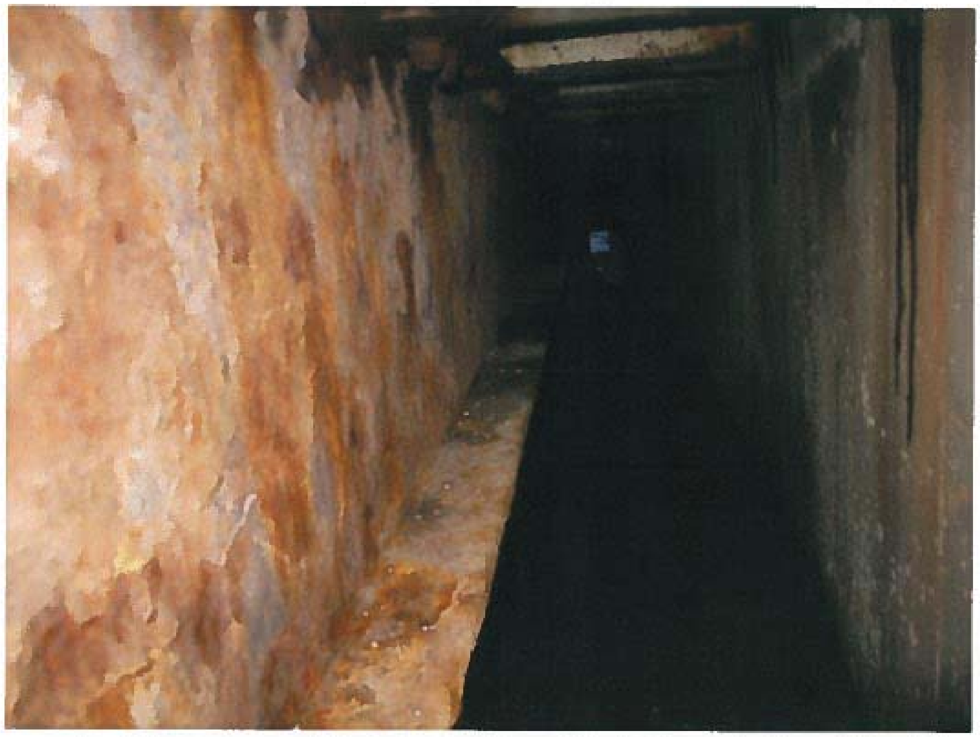
Photograph 28 – Perforation and section loss at stringer S8 between panel points 2W and 3W.

Photograph 29 – Typical condition of stringer/decking beams and grating. Note: coating break down and light to medium corrosion.

Photograph 30 – Typical condition of stringer/deck beams and grating. Note: the welded reinforcement to the bottom flange of the floor beam.

Photograph 31 – Typical condition of floor beams. Note: Welded reinforcement at the bottom flange.

Photograph 32 – Typical condition of connection angles of the floor beam to the lower chord.



Photograph 33 – Inside face of the end floor beam at east abutment.

Photograph 34 – Typical condition of horizontal bracing.

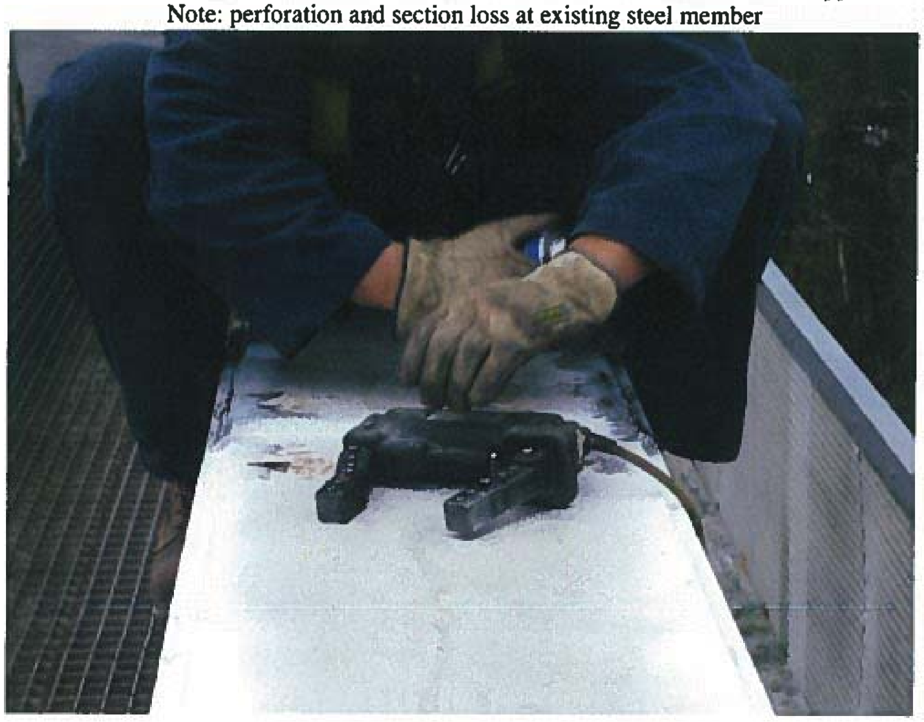
Photograph 35 – Typical condition of horizontal bracing at panel point connection. Note: Medium to severe corrosion.

Photograph 36 – Typical condition of structural steel at sidewalk overhang at east approach.

Photograph 37 – Typical condition of structural steel at sidewalk overhang at east approach. Note: Medium to severe corrosion

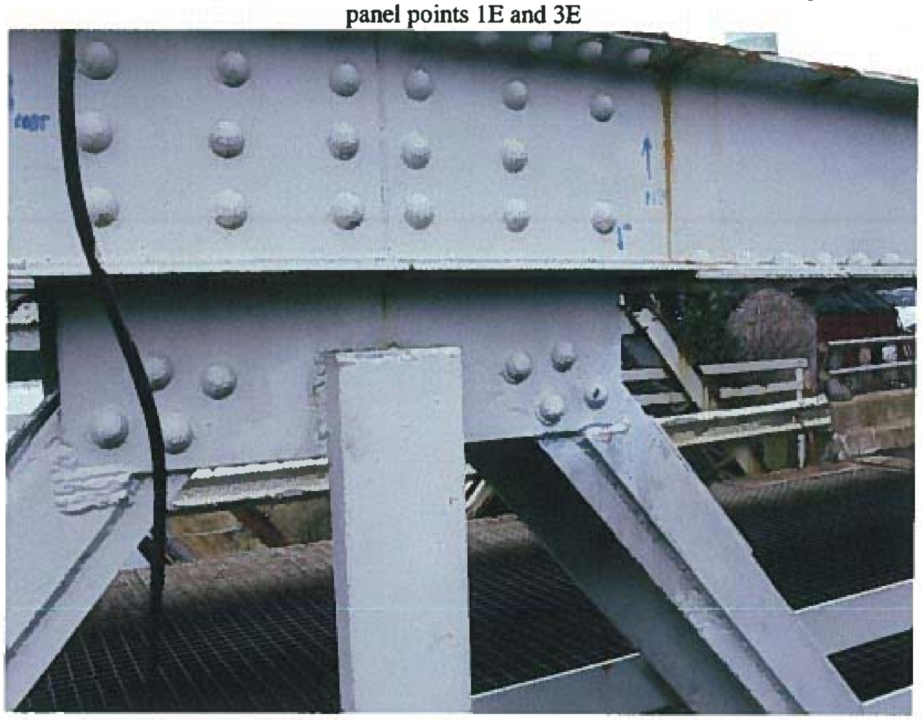
Photograph 38 – Welded reinforcement to existing transverse steel member at the sidewalk near the east abutment.

Photograph 39 – Welded reinforcement to existing transverse steel member at east app. sidewalk. Note: perforation and section loss at existing steel member



Photograph 40 – Magnetic particle testing of welded reinforcement at south truss top chord between panel points 1E and 3E

Photograph 41 – Magnetic particle testing of welded reinforcement at south truss top chord between panel points 1E and 3E

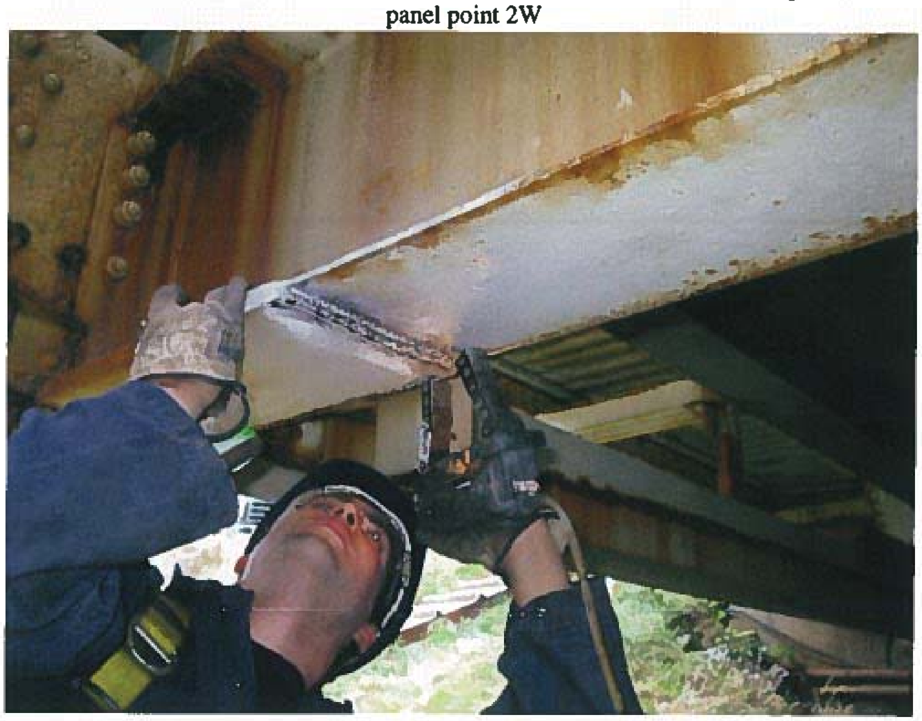


Photograph 42 – Magnetic particle testing of welded connection at the top truss diagonal member.

Photograph 43 – Crack in weld at welded reinforcement at panel point 5E south truss bottom chord.

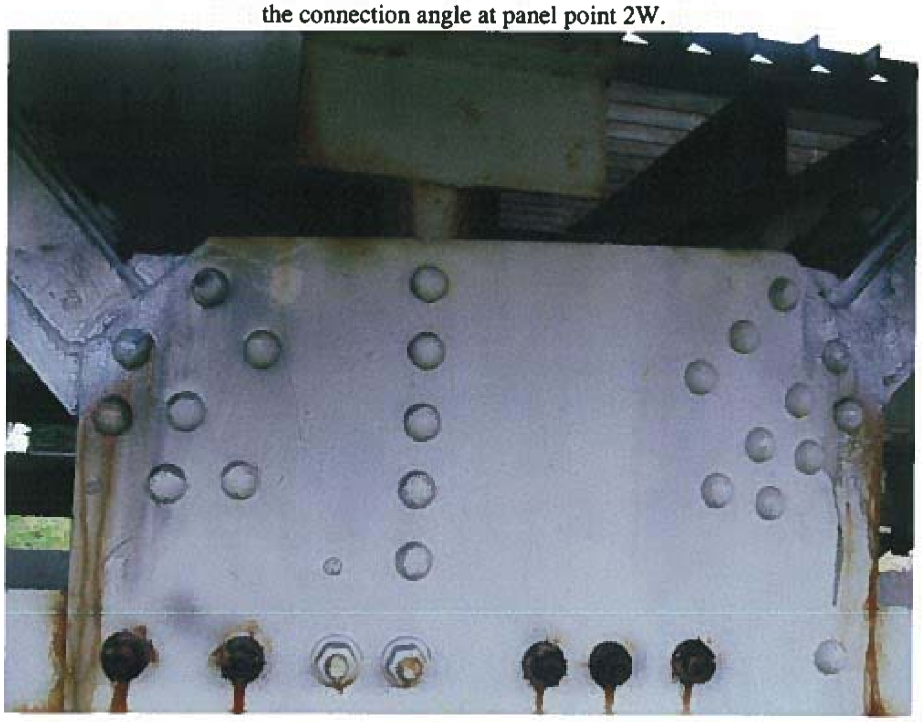
Photograph 44 – Crack in weld at welded reinforcement at panel point 5E south truss bottom chord.

Photograph 45 – Magnetic particle testing of welded reinforcement at bottom flange of floor beam at panel point 2W

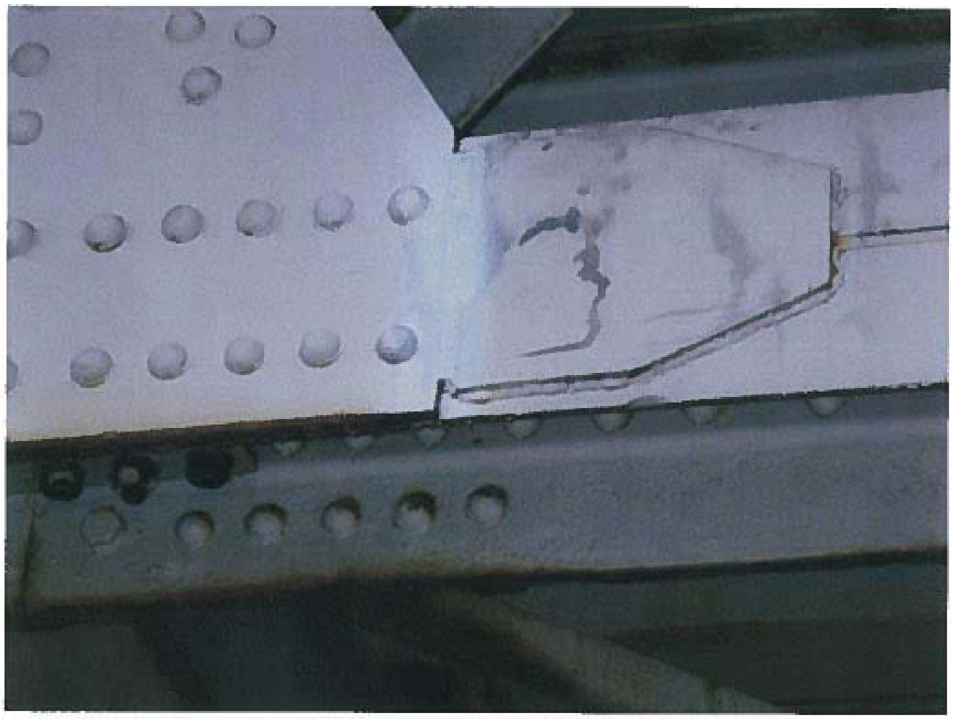


Photograph 46 – Magnetic particle testing of welded reinforcement at bottom flange of floor beam at panel point 2W

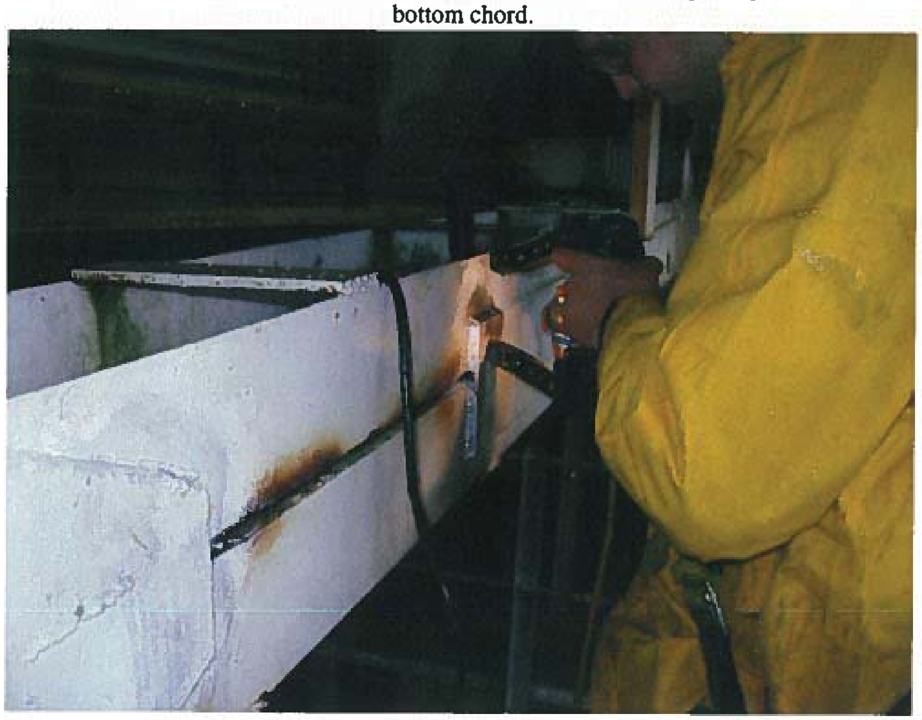
Photograph 47 – Magnetic particle testing of welded reinforcement at the web of the floor beam to the connection angle at panel point 2W.



Photograph 48 – Magnetic particle testing of welded reinforcement at the diagonal members at panel point 2W.



Photograph 49 – Magnetic particle testing of welded reinforcement at panel point 2E south truss bottom chord.



Photograph 50 – Magnetic particle testing of welded reinforcement at south truss bottom chord between panel points 4E and 5E.

APPENDIX C

Mechanical and Electrical Inspection

ST. PETER'S CANAL BRIDGE
MECHANICAL AND ELECTRICAL ASSESSMENT
MCCORMICK RANKIN CORPORATION

Byrne Engineering Inc.
Structural · Mechanical · Electrical

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Byrne Reference: 209280
December 2009

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December 17, 2009 - REVISED January 6, 2010, May 14, 2010

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P9

Attention: Ubaid Khan

Reference: St. Peter's Canal Bridge
Mechanical and Electrical Assessment REV3
Byrne Ref.: 209280

We are pleased to submit for your information, the report on the condition assessment of the St. Peter's Canal Bridge conducted by Byrne Engineering Inc.

If you have any questions on the following, please do not hesitate to contact Robert Mieloo or Apurva Patel at (905) 632-8044.

Yours truly,

BYRNE ENGINEERING INC.

Robert W. Moffat

Robert W. Moffat, P.Eng
Engineering Manager

ABP/acl

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APPENDIX A - CONSTRUCTION COST ESTIMATES

APPENDIX B – PHOTOGRAPHS

APPENDIX C – FLUID SPECIFICATION

EXECUTIVE SUMMARY

Byrne Engineering Inc. (Byrne) has conducted a thorough investigation to examine the current condition of the mechanical and electrical elements of the St. Peter's Canal Bridge. The following report presents the findings of the inspections, recommended repairs and the associated estimated engineering and construction costs.

Inspections

Inspections were performed by Byrne personnel on November 26, 2009. The inspections included identifying areas of mechanical and electrical concern. These areas were documented in field notes and photographs.

Recommended Repairs

- Replace tail wedge mechanism and actuators.
- Replace all bogey wheels and tail wheels.
- Replace bogey wheel and tail wheel shim packs with machined blocks. This will ensure that the shims do not swell and cause bogey wheel misalignment.
- Replace center slewing cylinders. Ensure that the new cylinders have stainless steel rods due to pitting issue (noted from conversations with bridge operator).
- Refurbish center slewing cylinder mounts and yokes.
- Replace hydraulic fluid with environmentally friendly alternative.
- Install appropriate hydraulic support hardware to adequately support all hydraulic lines.
- Traffic gate gearboxes are noisy; however, no other issues were noted. Based on Byrne's previous experience with refurbished gates, it is recommended that traffic gates should be replaced.
- Install sight glass to indicate center bearing oil level.
- Replace junction boxes and terminal boxes at center pier. These will also be relocated to the vertical face of the center pier.
- Replace flexible cabling.
- Relocate control panel to fixed abutment (west).

1 INTRODUCTION

The St. Peter's Canal Bridge is a single lane; single span bobtail swing bridge structure located over the St. Peter's Canal in St. Peter's, Nova Scotia. The bridge was originally constructed sometime in the early 1900's and in 1936 was moved from Southern Ontario to its current location.

The purpose of the report is to present the current condition of the bridge elements as examined during the inspections conducted by Byrne personnel. Recommended repairs and rehabilitation measures have been determined and are outlined in the report. The report also presents the associated engineering and construction cost estimates.

A detailed breakdown of the estimated engineering and construction costs is located in Appendix A.

Relevant photographs can be found in Appendix B.

2 BRIDGE DESCRIPTION

The bridge consists of a 40 m movable span over the St. Peter's canal. The center pier is located on the west side of the canal. The span is constructed of two steel plate truss girders, a single sided wood deck sidewalk and steel grating deck. A large concrete counterweight is located on the west side of the movable span to compensate for the uneven span lengths.

Entry to the centre pier is by way of a ladder located at the north side of the center pier. This provides access to the center pivot mechanical equipment.

The bridge is swung by two hydraulic cylinders mounted on the center pier. The valves that control the cylinders are located in a control panel located at the road deck level just above the center pier. There are two end wedges located at the east end of the bascule span which lock and level the movable span. The control valves for the wedges are also located in the control panel.

There are a total of two electric motor operated traffic gates for controlling traffic during bridge operation.

The bridge is opened roughly 300 times a year.

3 BRIDGE INSPECTION

3.1 Operator / Maintenance / Technician Interview

The operator reports several items which are noteworthy:

During the operator interview, the operation sequence for the bridge was discussed and described to Byrne representatives. The operation of the bridge is fully manual and there are no indicating limits for completion of certain sequences in the operation. The sequence of operation is as follows:

- Bridge opening
  - Switch on pre-emptive switch located in machinery house. Once this switch is selected to the on position, it turns the traffic lights red and after a 30 second delay, it provides power to the bridge.
  - The operator then walks to the operator panel located on the movable span of the bridge.
  - Lower the traffic gates using the switches at the operator's panel.
  - Switch on the hydraulic power unit from the operator's panel.
  - Using the manual hydraulic valve on the panel, disengage the tail wedges. When the wedges are disengaged, a "wedges retracted" lamp should come on. It was noted that the lamp was not functional at the time of inspection and that the operators simply listen to the hydraulic valve to know when the wedges have been retracted.
  - Using the manual hydraulic valve, open the bridge. Due to the manual control, the bridge slewing speed towards the end of travel must be controlled by the operator so that the bridge does not impact the end stop.
  - The navigation light on the canal wall north of the bridge will turn from red to green when the bridge is fully opened.
- Bridge closing
  - When clear to do so, using the manual hydraulic valve, close the bridge. Due to the manual control, the bridge slewing speed towards the end of travel must be controlled by the operator so that the bridge does not impact the end stop.
  - Once the bridge is in line with the roadway, engage the end wedges. The hydraulics have been set up so that the south grooved wedge is engaged first in order align the span before the flat north wedge is engaged. The difference in sound of the fluid flowing through the control valve will indicate that the wedges are fully engaged.
  - Turn off the hydraulic pump
  - Raise the traffic gates
  - Ensure the roadway is clear for traffic and turn off the pre-emptive switches to turn the traffic lights green and remove power from the bridge equipment.

3.2 Mechanical

End Wedges

The end wedges are located at the east end of the movable span and consist of a wedge that is driven by a hydraulically driven piston-crank mechanism (Photograph 1). The north wedge is the span locating wedge, it has a groove with a mating key on the sliding wedge, and this wedge is actuated first to position the movable span in the correct position (Photograph 2). The south wedge is then actuated to assist in leveling the span. The mechanisms for actuating the wedges are functional, however, the components are corroded and some mechanical damage was noted (Photographs 3 & 4). The hydraulic cylinders that actuate the wedges are fitted with boots to protect the exposed rods; however, when the boot was pulled back, pitting was noted on the rods. A turnbuckle is used to act as the link connecting the wedge to the mechanism; this should not be used in this application because it is not designed to apply a push force. The wedge housings / guides are corroded and replacement of the entire wedge system is recommended. When replacing the wedges, the structure that supports the mechanisms must be inspected to ensure that it will be able to support the loads applied by the wedges.

Main Drive Machinery

The main drive machinery consists of two hydraulic cylinders that are mounted to the center pier with the rod yoke attached to the bridge structure (Photographs 5 & 6). The cylinders are mill duty cylinders and are in good condition, no leaks were noted. The south east cylinder was replaced in 2008 due to rod pitting. The operator noted that the exposed rod is subject to roadway debris and salt in the winter months due to the open road deck grating.

Center Bearing

The center bearing was inspected and polished in 1998. The bearing was noted to be in good condition during the inspection. The bearing housing retains oil and no leaks were noted. Regular maintenance is evident and the bearing was filled with oil. The operator mentioned that a sight glass would be beneficial to indicate the oil level inside the bearing housing. An oil overflow line is located on the north side of the bearing housing; the line was traced and found to drain on the west side of the concrete pier. This drain is not environmentally acceptable (Photographs 7 & 8).

Hydraulic Machinery Room

The hydraulic power unit is located in a machinery house on the west side of the canal. The machinery was installed in the late 1970's and was noted to be in good working condition (Photograph 9 & 10). The operation of the hydraulic unit was observed and no issues were noted. The hydraulic lines were replaced 25 years ago with stainless steel lines with Swagelok fittings. The lines were visually inspected and appear to be in good condition. The filters were reported to be changed regularly. The observation was made that there is no fluid drip tray installed under the power unit. The drip tray is vital to ensure that if there is a leak, the hydraulic fluid is contained. The control valves for the wedge and slewing cylinder operation are located in the control panel on the movable span. All observed hydraulic fluid lines were observed to be secured in place with uni-strut style mounting hardware; this is not appropriate for hydraulic lines and proper hydraulic support hardware should be installed. It was noted that the bridge operators were interested in replacing the current hydraulic fluid with an environmentally friendly option in the event of a leak; a specification for a fluid has been included with this report in Appendix C however a system check must be completed to ensure that there are no aluminum components currently installed. Proper flushing of system is required as part of this process.

Control System

The bridge is manually operated through a control panel mounted on the movable span at the road deck level directly above the center pier. The panel contains the controls for the traffic gates, hydraulic unit power control, and the manually operated hydraulic valves for cylinder operations. The panel also contains a wedge retracted indicator lamp which was not functional at the time of the inspection.

Traffic Gates

There are two electrically driven traffic gates at the east and west abutments for traffic control. The gates are approximately 30-40 years old and regular maintenance is evident. Traffic gate operation was observed and the gearboxes were noted to be noisy; however no other issues were noted. Replacement is not recommended at this time however, if the gates fail in the future, Byrne's experience is to replace rather than repair the unit.

Bogey wheels and track

The bogey wheels and track stabilize the bridge during the slewing operation. There are a total of four bogey wheels mounted to the structure of the bridge, there is a journal bearing contained in the wheel with grease fittings located on the main shaft. All four bogey wheels rotate with little resistance and regular greasing is evident. The track is a standard rail that is mounted with clips to the center concrete pier. The rail is original to the construction of the bridge and is showing signs of wear. The bogey wheels were reported to have been replaced but are also showing signs of wear. The bogey wheels have been shimmed over time and some of the wheels contain many shims. These shims were also observed to be corroding. Over time, the corrosion between the shims might cause the shims to separate and ultimately misalign the wheels (Photographs 11 & 12).

End Stops

There a total of two end stops on the bridge, one located at the east end for the bridge closed stop and one at the center pier for the bridge open stop. Each of the end stops contain a rubber pad assembly to act as a cushion in the event that the bridge were to come into contact with either of the end stops. The steelwork at both end stops was corroded but still functional; the rubber however was cracked and deterioration due to sunlight (Photographs 13 & 14).

Tail wheels and receivers

There are two tail wheels at the west end of the movable span; they support the west side of the bridge when in the closed position. The tail wheels rest on two steel receivers that are profiled to contain and guide the wheels. The steel receivers are anchored to the fixed west abutment. The wheels, wheel housings and receivers are worn and corroded. Over time, the corrosion between the shims might cause the shims to separate and ultimately misalign the wheels (Photograph 15).

Bridge balance

Due to the "bobtail" design, the bridge has been fitted with a large concrete counterweight at the west end of the bridge; this counterweight also acts as the road deck. Some time after erection, a sidewalk was added to the south side of the movable span. To counterbalance the weight due to the sidewalk and associated structure, concrete blocks were added to the north truss girder. The blocks were mounted to the bottom chord of the truss girder with threaded rods. For fine tuning, there are steel plates placed on the bottom chord of the south truss girder. The operator indicated that the bridge balance and bogey wheel clearances are checked on a yearly basis. The bridge was well balanced during the assessment (Photographs 16 & 17).

3.3 Electrical

The St. Peter's Bridge is a completely manual operated bridge using manual hydraulic valves with location proximity switches used for indicator lights.

Main Electrical Distribution

The main electrical distribution in the machinery house appears to be in good condition. It provides power to the hydraulic power pack, the operator's panel and the traffic gates.

Junction and Terminal Boxes

The field junction and terminal boxes are in very poor condition. There were a combination of stainless steel boxes and PVC type junction boxes. The exterior of the stainless boxes were in fair condition; the internal and strain relief connection components were in very poor condition. Most strain relief connection components to the stainless steel were stainless, but there were a few non stainless. In these areas, the internal lock nuts were not stainless and appeared corroded; there were no visible seals in the connectors. The moisture in the boxes would be due to the corroded lock nut and missing rubber seals (Photograph 18, 19).

The PVC junction box near the traffic gate on the west (St. Peter's) side of bridge has been loosened from its anchoring.

It is recommended that the junction boxes be replaced and repositioned to be on the side of the center pier. This would allow ease of maintenance and would also provide protection from standing water. The boxes should be equipped with water vents in the bottom to allow moisture to escape. Boxes should be protected with drip covers.

Control Panel

The operator control panel is located on the bridge. The operator control panel faces the traffic side of the bridge. It was noted by the operators that it would be preferred if the operators panel was relocated to a safer location. It is recommended that the control panel be relocated to the west fixed abutment, however an elevated steel platform may be required to provide line of sight (Photograph 20).

Cables

The cables exiting the machinery house to the center pier appeared to be in fair condition.

The flexible cables from the junction / terminal boxes are in very poor condition and should be replaced.

The junction boxes adjacent the traffic gates (St. Peter's side) should be removed and the cables should be direct wired to the traffic gate. (Photograph 21)

Proximity Sensors

The proximity sensors only provide indication of bridge location. There are two proximity switches located on the center pier and one on each of the wedges.

The proximity switches were all in good condition. (Photograph 22)

Indication Lights

The indication lights did not work when the bridge was inspected.

The navigational lights did appear to be working. The lights change from red to green based on the proximity switch and a relay in the junction box, both located on the center pier.

4 RECOMMENDATIONS

Recommended rehabilitation work of the mechanical / electrical components of the St. Peter's Canal Bridge has been considered in the following estimate. Priorities have been determined and noted beside each item:

- Replace tail wedge mechanism and actuators (1-2 years).
- Replace all bogey wheels and tail wheels (1-2 years).
- Refurbish center slewing cylinders. Ensure that the new cylinders have stainless steel rods due to pitting issue noted from conversations with bridge operator (2-5 years).
- Refurbish center slewing cylinder mounts and yokes (1-2 years).
- Replace hydraulic fluid with environmentally friendly alternative (see Appendix C) (ASAP).
- Install appropriate hydraulic support hardware to adequately support all hydraulic lines (ASAP).
- Install sight glass to indicate center bearing oil level and reroute oil overflow drain to a holding tank (ASAP).
- Replace junction boxes and terminal boxes at center pier. These will also be relocated to the vertical face of the center pier (ASAP).
- Replace flexible cabling (ASAP).
- Relocate control panel to fixed abutment (west) (1-2 years).
- Replace cabling to the west traffic gate. This will eliminate the unsecured junction box (ASAP).
- Install hydraulic drip tray to catch fluid in the event of a spill (ASAP).
- Remove and replace the control panel, relocation to the west approach is recommended. Due to line of sight issues the panel may have to be mounted on an elevated steel structure. The new panel will also include indicator lamps to show the operator the position of the bridge at the nearly closed and nearly open positions (2-5 years). A temporary measure of moving the control panel so it faces the sidewalk should be completed as soon as possible to enable the operator to work from the sidewalk rather than the road.

5 CONSTRUCTION COST ESTIMATES

The total construction cost estimates for repairs to the bridge are as follows:

Appendix A shows a breakdown of this estimate by individual item.

Limitations

Cost estimates are generally based on one of the three methods:

- Quotations from suppliers.
- Extrapolation of cost or quotes for previous work.
- Direct estimation by the consulting engineer.

While every effort has been made to establish the appropriate costs, it must be recognized that the figures represent an estimate. The actual costs may vary even with standard types of construction. For non-standard work, such as bridge rehabilitation work, the variation may be greater.

The estimates are therefore subject to change and are contingent upon factors over which we have no control. The estimates are not guaranteed as to accuracy. Exact costs will be determined only when tenders have been received for the project.

Estimates are budgetary, and do not constitute an offer to perform the work. There is no allowance for taxes, inflation or material and labour market conditions.

APPENDIX A

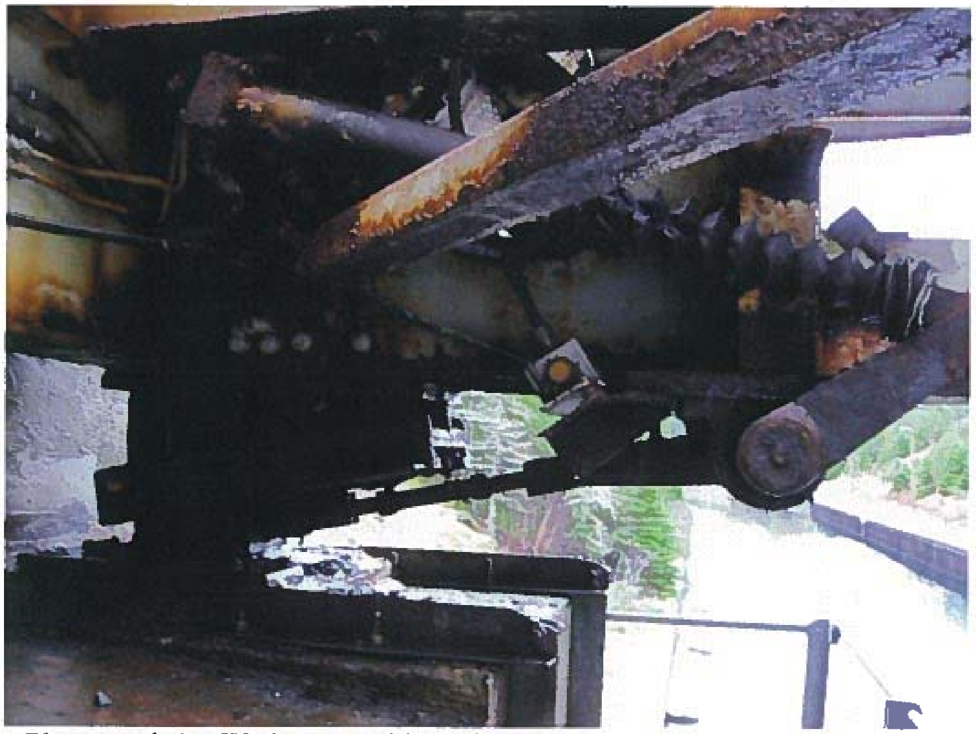
CONSTRUCTION COST ESTIMATES

Rehabilitation Estimate Summary - St. Peters Canal Swing Bridge		
	Description	**Cost**
1	Immediate repairs	$76,000.00
2	Wedge system replacement	$256,000.00
3	Control panel relocation	$261,000.00
4	Bogey wheel replacement	$178,000.00
TOTAL Including 20% Contingency		**$926,000.00**

Note: The above cost estimates do not include engineering support for the construction, tendering or commissioning phases.

APPENDIX B

PHOTOGRAPHS



Photograph 1 – Wedge assembly at the south-east end of the movable span.

Photograph 2 – North east wedge, note locating feature in wedge and wedge receiver.

Photograph 3 – Wedge crank arm

Photograph 4 – South east wedge bent crank arm

Photograph 5 – North slewing cylinder

Photograph 6 – South slewing cylinder

Photograph 7 – Center bearing oil drain line

Photograph 8 – Center bearing oil drain line (pipe on the left side)

Photograph 9 – Hydraulic power unit 1

Photograph 10 – Hydraulic power unit 2

Photograph 11 – South bogey wheel

Photograph 12 – East bogey wheel, note number of shims

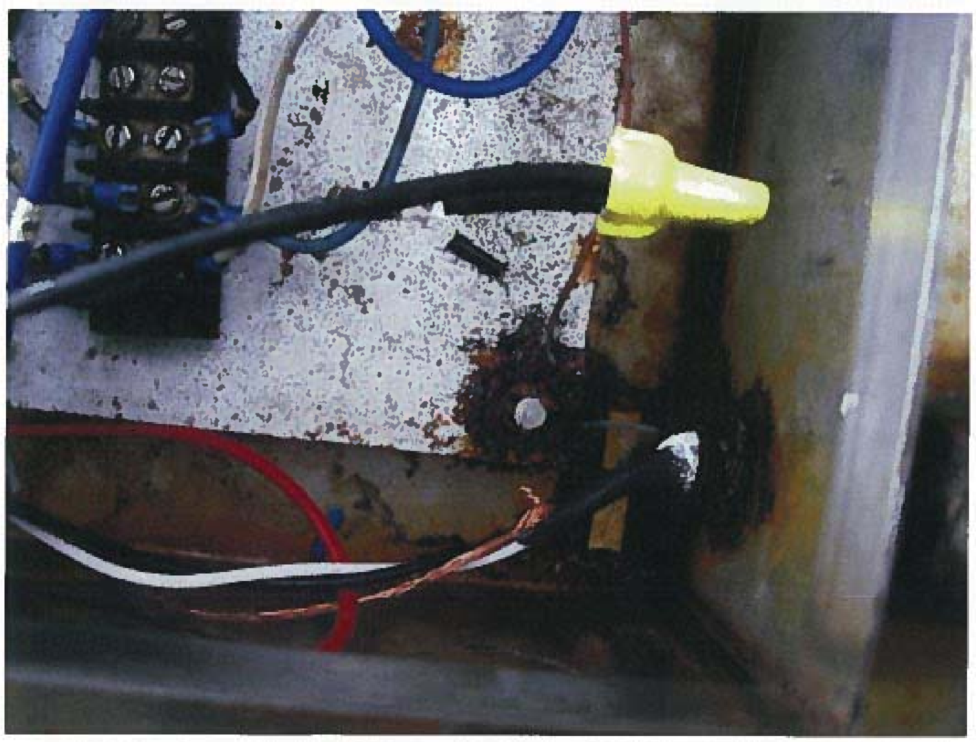
Photograph 13 – Bridge open end stop, note that rubber pad is sandwiched in between steel plates

Photograph 14 – Bridge closed end stop located on west fixed abutment

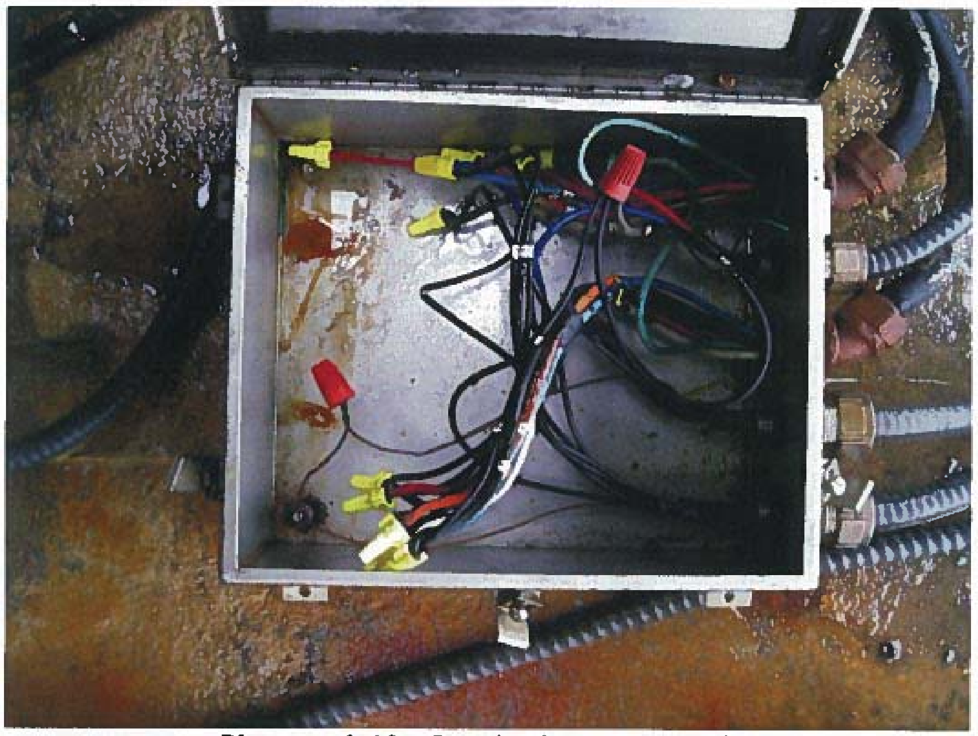
Photograph 15 – Tail wheel

Photograph 16 – Concrete weights attached to bottom chord of north truss

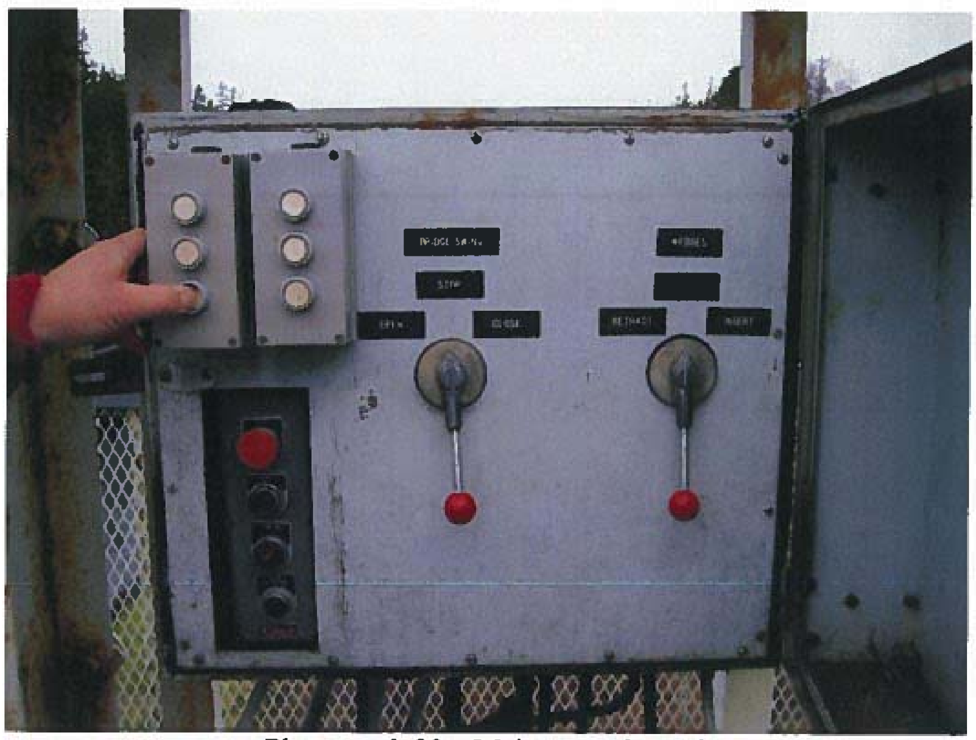
Photograph 17 – Counterweight box for steel counterweights for fine adjustment



Photograph 18 – Junction and Terminal Boxes



Photograph 19 – Junction box at center pier



Photograph 20 – Main control panel

Photograph 21 – Junction box at west traffic gate

Photograph 22 – Bridge open proximity sensor

APPENDIX C

FLUID SPECIFICATION

GOING GREEN

Power Flo

Environmentally Approved Hydraulic Fluid

Power Flo is a premium, high pressure, water soluble, environmental hydraulic fluid, which provides a more responsible environmental option than conventional mineral oil or vegetable oil based hydraulic fluids. In applications where hydraulic piping may rupture and spray, or leak hydraulic fluid into sensitive environments, Power Flo is readily biodegradable, and dissolves entirely, protecting fish and fowl from oily slicks. Power Flo's technical performance is equal to or better than premium mineral oil hydraulic fluids in service.

Recommended Uses

Power Flo is recommended for use in precision industrial and mobile hydraulic systems operating at system pressures up to 7000 psi at 50 °C (Note: higher operating pressures and temperatures may be possible).

Power Flo is recommended for use in stationary or mobile hydraulic systems operating in sensitive environments where spills or leaks may occur.

Power Flo high pressure hydraulic fluid is a Group 1 Factory Mutual approved fire-resistant fluid.

Features	Benefits
• Readily Biodegradable	• >85% in 28 days. Incidental drips or leaks degrade in the environment leaving no lasting effects.
	• Reduced environmental liability, reduced clean-up costs
• Low order of Toxicity	• Rated by USFW as "Essentially Non-Toxic."
	• Not harmful to fish or wildlife
• Water Soluble	• Easy Clean-up
	• Will not form slicks or sheens when released into water. Dissolves, disperses and degrades. Will not foul shorelines or plant life.
• High Viscosity Index, Low Pour Point	• Resists viscosity change with temperature. Works better than mineral oil at operating temperature extremes. Pour point of -63 degrees C.
• Not WHMIS controlled	• Worker Acceptance
• NON TDG controlled	• Lower Freight Costs
• Eco-Logo Approved	• Your evidence of due diligence choosing a Government of Canada approved environmental product
• Fire Resistant	• Worker and Asset Protection

Forsythe Lubrication Associates Ltd. 120 Chatham St. Hamilton, Ontario, Canada L8P 2B5
Phone: (905) 525-7192 Fax: (905) 525-7024 Toll Free: (800) 363-2759

Typical Technical Properties

Property	Method	Typical
Colour & Appearance	Visual	Clear, Blue
Viscosity, cSt @ 40 °C	ASTM D-445	46
Water by Karl Fischer Titration, %	ASTM D-1744	39
Pour Point, °C	ASTM D-97	-63
pH	FLA 003	9.0
Reserve Alkalinity, (mL 0.1N HCl to neutralize 100mL of fluid to pH 5.5)	FLA 011	180
100 Hour Pump Stand Test, mg wear / hour	ASTM D-2882	0.10
100 Hour Vickers Pump Stand Test, mg wear /hour	20-VQ-5	0.13

Notes

20-VQ-5 Pump Stand Test Conditions:

Pump Pressure	3000 psi
Pump Speed	1200 rpm
Flow Rate	5 GPM
Temperature	65°C

The Vickers 20-VQ-5 Pump Stand Test has been developed to provide hydraulic fluid performance data in combination with ASTM D-2882.

Packaging

Power Flo Environmental hydraulic fluid is available in 5 gallon pails or 55 gallon drums.

CONTACT INFORMATION: Phone: (905) 525-7192 or (800) 363-2759
Forsythe Lab: flalab@forsythe.on.ca
Ron Arbour (Tech Service Rep): rarbour@forsythe.on.ca
Troy Olmsted (Tech Service Rep): tolmsted@forsythe.on.ca
Helen Stanton (Order Desk): (905) 525-7192
www.forsythe.on.ca

APPENDIX D

Cost Estimates

St. Peter's Canal Bridge
Nova Scotia

Cost Estimate for Structural Steel Rehabilitation

Item	Description	Unit	Est. Qty	Unit Price	Total
1	Mobilization	L.S.	-	$ -	$ 20,000.00
2	Traffic Control	L.S.	-	$ -	$ 25,000.00
3	Field Office	L.S.	-	$ -	$ 20,000.00
4	Access to Work	L.S.	-	$ -	$ 65,000.00
5	Remove and Replace Rivets/Bolts	each	300	$ 150.00	$ 45,000.00
6	Remove and Replace Lacing Bars	each	100	$ 350.00	$ 35,000.00
7	Reinforce Existing Gusset Plates	each	6	$ 6,000.00	$ 36,000.00
8	Additional Reinforcement for Various Structural Components	each	20	$ 4,000.00	$ 80,000.00
9	Supply Miscellenous Steel	tons	5	$ 3,000.00	$ 15,000.00
10	Hourly Rates of Iron Workers and Welders	each	200	$ 130.00	$ 26,000.00
10	Steel Repairs at South Sidewalk	L.S.	-	$ -	$ 15,000.00
11	Repair of Cracked Weld	L.S.	-	$ -	$ 2,000.00
		TOTAL			$ 384,000.00
		CONTINGENCY (20%)			$ 76,800.00
		Subtotal			$ 460,800.00
		TOTAL (Excluding Taxes)			**$ 461,000.00**

Note: The cost in the above table doesn't include electrical and mechanical components of the bridge rehabilitation work. For electrical and mechanical rehabilitation work cost estimates, please see Appendix C.

St. Peter's Canal Bridge
Nova Scotia

Cost Estimate for Cleaning and Coating of Structural Steel

Item	Description	Unit	Est. Qty	Unit Price	Total
1	Mobilization	L.S.	-	$ -	$ 20,000.00
2	Traffic Control	L.S.	-	$ -	$ 50,000.00
3	Field Office	L.S.	-	$ -	$ 20,000.00
4	Worker and Environmental Protection	L.S.	-	$ -	$ 150,000.00
5	Containment Requirements	m^2	1300	$ 100.00	$ 130,000.00
6	Cleaning and Coating of Existing Steel	m^2	1300	$ 150.00	$ 195,000.00
		TOTAL			$ 565,000.00
		CONTINGENCY (20%)			$ 113,000.00
		Subtotal			$ 678,000.00
		TOTAL (Excluding Taxes)			**$ 678,000.00**

Note: The cost in the above table doesn't include electrical and mechanical components of the bridge rehabilitation work. For electrical and mechanical rehabilitation work cost estimates, please see Appendix C.

Public Works and Government Services Canada
Travaux publics et Services gouvernementaux Canada

HIGHWAY BRIDGE EVALUATION FOR PARKS CANADA IN ATLANTIC PROVINCES

St. Peters Canal National Historic Site
Volume 2 of 2
Bridge Evaluation Report

MRC
Consultants in Transportation

February 2007

PARKS CANADA

ST. PETERS CANAL

NATIONAL HISTORIC SITE

Report Prepared By:	Report Reviewed By:
ASSOCIATION OF PROFESSIONAL ENGINEERS OF N.S. 2007 Date: Feb 28/07 K. Albazi, P.Eng. 16008 License To Practice	ASSOCIATION OF PROFESSIONAL ENGINEERS OF N.S. 2007 Date: Feb 28/07 D. G. Dixon, P.Eng. 16009 License To Practice
Karam Albazi, P. Eng. Senior Bridge Engineer	Doug Dixon, P. Eng. Senior Bridge Engineer

VOLUME 2 of 2

BRIDGE EVALUATION REPORT

February 2007

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Appendix Bridge Evaluation Report

File: W:\6k\6572 Parks CDA Atlantic Provinces Bridge & Culvert Ins.-Eval\6572.300 Structural\6572.304 Reports\6572 BRIDGE EVALUATION PROCEDURE ST. PETERS CANAL.doc

BRIDGE EVALUATION PROCEDURE

The bridge evaluation procedure for Atlantic Provinces Parks Highway Bridges is summarized as follows:

1. COLLECTION OF DATA

The availability of contract drawings and shop drawings for the bridge was checked. Based on the drawings, the field inspection staff was informed about the necessary information to be collected from site. Various pieces of information about the bridge were obtained, i.e. determination of bridge type, number of spans, time of construction, materials of construction etc. The information was also obtained with regard to rehabilitation of the bridge. The inspection data, report, and photographs of the bridge were also collected.

The bridge dimensions and the component sizes were taken from the drawings subject to confirmation by the inspection. When the drawings were not available, measurements of the components recorded during inspection were used depending upon the type of material.

Once the above information was assembled, the following procedure was followed.

2. MATERIAL STRENGTHS

The contract and shop drawings provided by the department were first checked for information about material strengths. If the drawings did not provide adequate information about the material strengths "Canadian Highway Bridge Design Code" (CHBDC), CAN/CSA-S6-00 clause 14.6 provisions were used in the evaluation.

Based on provisions of this clause, the strengths of different types of steel were determined as follows:

Structural Steel

Date of Bridge Construction	Specified Fy, MPa	Specified Fu, MPa
Before 1905	180	360
1905 - 1932	210	420
1933 - 1975	230	420
After 1975	250	420

where Fy and Fu are yield and ultimate strengths respectively.

Reinforcing Steel

Based on the information available in the drawings about the grade of steel (structural grade, medium or hard), the yield strength fy, (MPa), of the reinforcing steel was determined as below:

Date of Bridge Construction	Grade Struct.	Medium	Hard	Unknown
Before 1914	-	-	-	210
1914 - 1955	230	275	345	230
1956 - 1978	275	345	415	275
After 1978				
- stirrups & ties	300	350	400	300
- remainder	300	350	400	350

Prestressing Steel

The tensile strength fpu, was taken as follows:

Date of Bridge Construction	fpu (MPa)
Before 1963	1600
Otherwise	1725

Concrete

CHBDC clause 14.6 recommends the use of following concrete strengths in the absence of any information in the drawings.

	fc´ (MPa)
Reinforced Concrete	
in substructure	15
in superstructure	20
Prestressed Concrete	25

When the above values in MRC's opinion, did not seem realistic, sample tests might be recommended to determine actual strengths in accordance with CHBDC clause A14.1.

3. MATERIAL DENSITIES

The following densities for various materials were used in evaluation in accordance with CHBDC clause 3.6:

Material	**Unit Weight KN/m^3**
Bituminous Wearing Surface	23.5
Concrete:	
Reinforced	24.0
Prestressed	24.5
Granular Soil	22.0
Wood:	
Hardwood	9.5
Softwood	6.0
Steel:	77.0

4. DEAD LOADS

The dead loads on various components of the bridge were calculated using the material densities in the previous section. Three types of dead loads were calculated:

D1: Dead loads of factory produced components, and cast-in-place concrete excluding decks

D2: All other dead loads (such as deck slab, curbs, railings, sidewalks etc.) except asphalt. Asphalt loads may be included if exact thicknesses are available

D3: Asphalt loads, where 90 mm of thickness is assumed in the absence of exact information

5. LIVE LOADS

The live load truck and lane loads were applied as per CHBDC 14.8.1 wherein, CL-1-W truck and corresponding lane loading is specified for level-1 evaluation. However, for bridges in the province of “New Brunswick”, CL-1-625-ONT truck and corresponding lane load were used for analysis. (See Figures)

Appropriate addition of “Dynamic Load Allowance” (DLA) was applied to the live load truck in accordance with CHBDC 3.8.4.5 as follows:

<u>DLA</u>

0.50 For deck joints

0.40 Where only one axle of the CL-W truck was used, except for deck joints

0.30 Where any two axles of CL-W truck, or axles 1, 2 and 3 were used

0.25 Where 3 axles of the CL-W truck, except for axles 1, 2 and 3, or more than 3 axles were used

When the results of analyses and resistance calculations showed that posting might be required, additional analyses using CL-2-W and CL-3-W trucks and corresponding lane loadings were performed.

Live load distribution factors for distribution of shear and moment were calculated based on provisions of CHBDC 5.7.1. The simplified method was used if the conditions set by CHBDC 5.7.1.1 were satisfied.

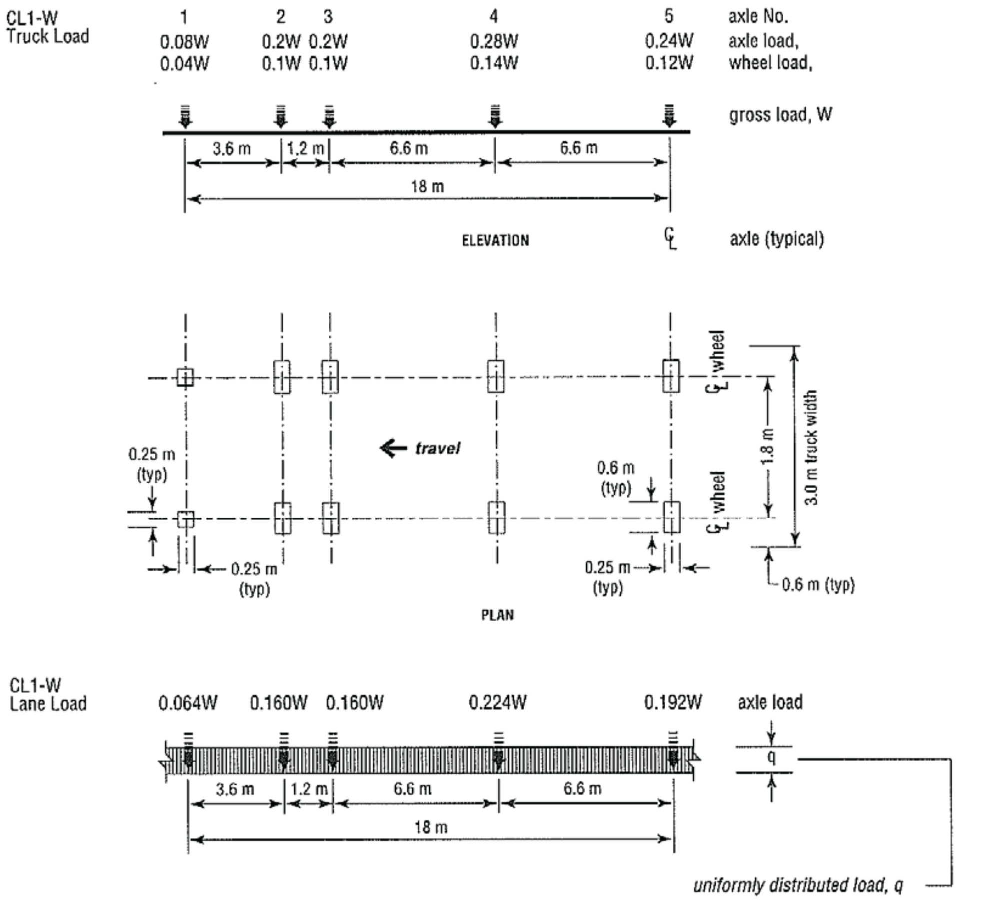
Live load for pedestrian bridge was taken in accordance with CHBDC 3.8.9 or as per manufacturer's drawing whichever was greater. Alternately, 80 KN maintenance vehicle load was also used in separate analysis and the controlling forces from the two analyses were used for evaluation.

6. LIMIT STATES

The requirements for ultimate, serviceability and fatigue limit states were checked as per clauses 14.4.1 and 14.18 of the CHBDC.

For concrete and wood components, and for steel components without fatigue prone details or fatigue related defects, only ultimate limit state was investigated. It would include factored combination of forces to find ultimate moments, shear forces etc. and full strengths (concrete strength or steel yield strength) of the materials was used in strength computation.

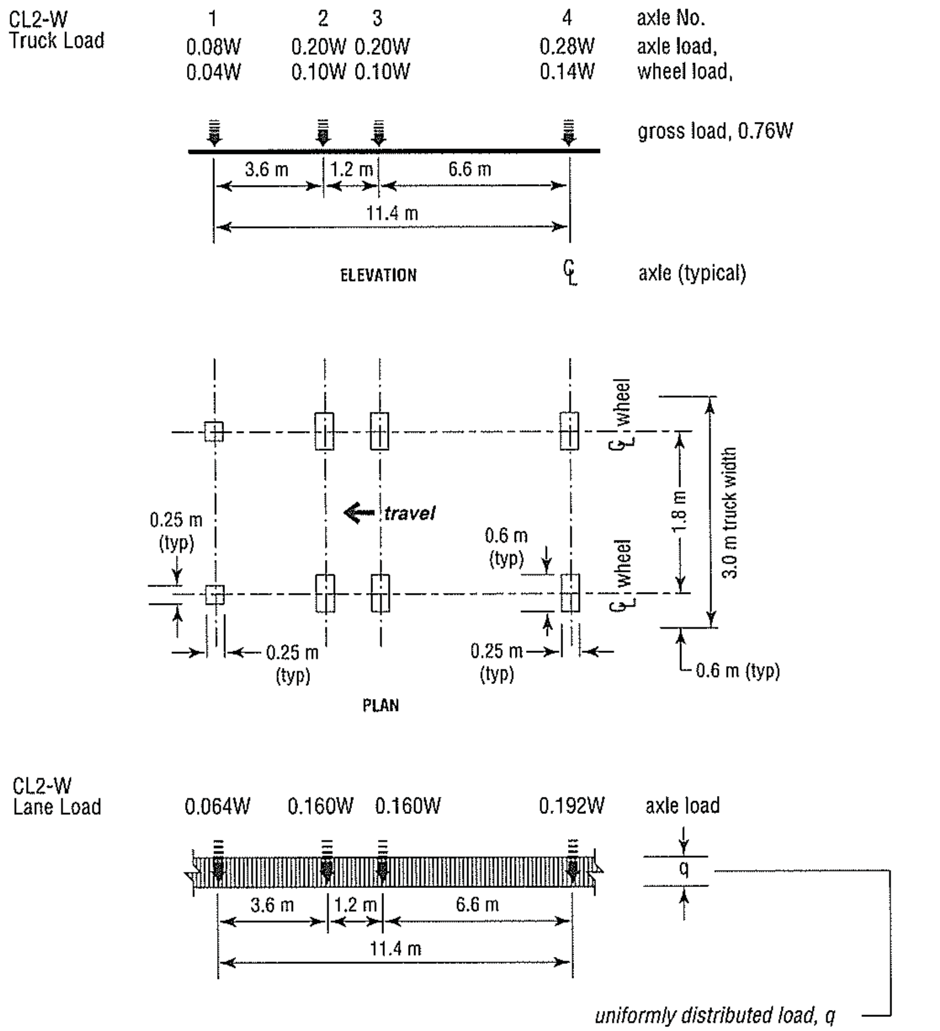
Level 1 Evaluation Loads with CL1-W Truck



Highway Class	A	B	C or D
q, (kN/m)	9	8	7

For definition of Highway Class, see Section 1.

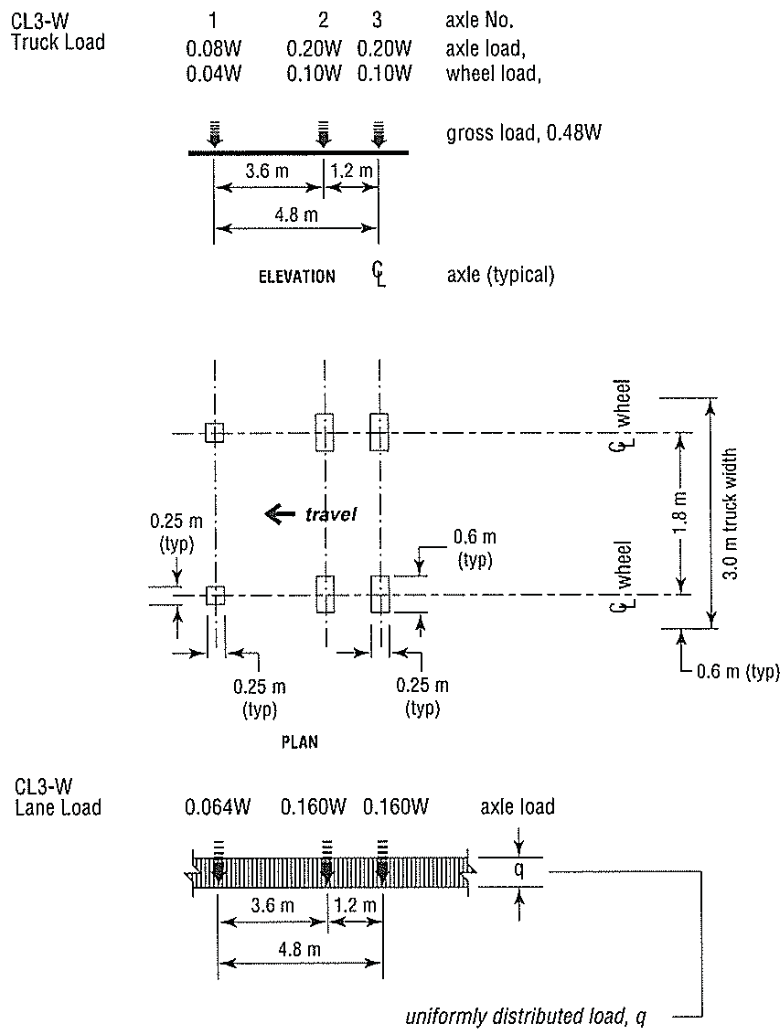
Level 2 Evaluation Loads with CL2-W Truck



Highway Class	A	B	C or D
q, (kN/m)	9	8	7

For definition of Highway Class, see Section 1.

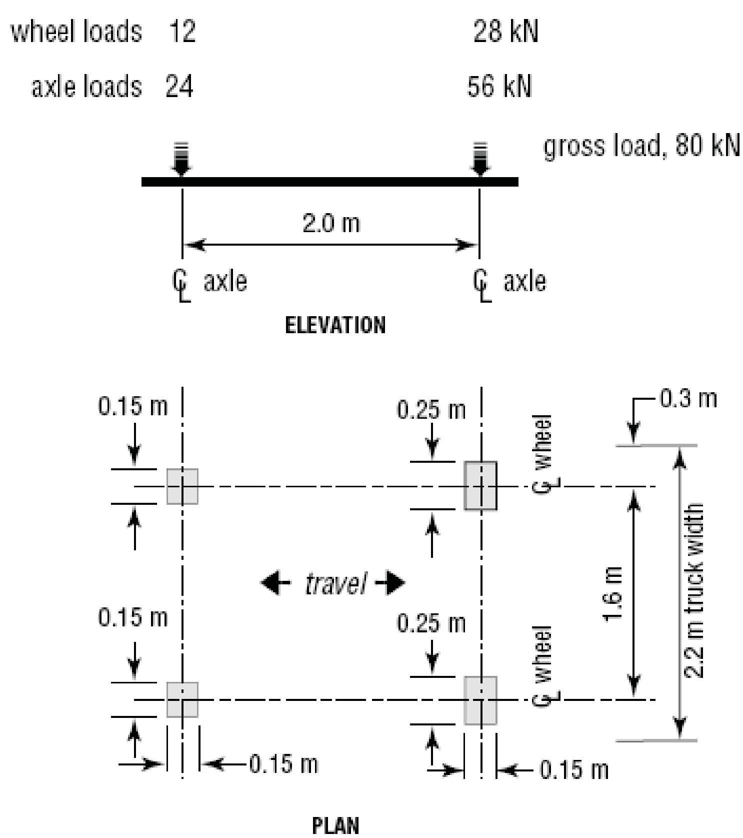
Level 3 Evaluation Loads with CL3-W Truck



Highway Class	A	B	C or D
q, (kN/m)	9	8	7

For definition of Highway Class, see Section 1.

Maintenance Vehicle Load



7. LOAD FACTORS

The load factors were applied to the different types of dead loads and live load in analyses for ultimate limit state evaluation. These load factors are based on the reliability of different types of loads and are therefore tabulated against the "Target Reliability Indices" as below:

Maximum Dead Load Factors, $\propto_D$

	Target Reliability Index, β								
Dead Load Category	2.00	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00
D1	1.03	1.04	1.05	1.06	1.07	1.08	1.09	1.10	1.11
D2	1.06	1.08	1.10	1.12	1.14	1.16	1.18	1.20	1.22
D3	1.15	1.20	1.25	1.30	1.35	1.40	1.45	1.50	1.55

Live Load Factors, $\propto_L$

	Target Reliability Index, β						
	2.50	2.75	3.00	3.25	3.50	3.75	4.00
All Spans	1.35	1.42	1.49	1.56	1.63	1.70	1.77

The target reliability index used in the above tables was obtained from CHBDC Table 14.11 (a) as under:

Target Reliability Index, β, for Normal Traffic, PA, PB, and PS Traffic

System Behaviour	**Element Behaviour**	**Inspection Level**		
		INSP1	**INSP2**	**INSP3**
S1	E1	4.00	3.75	3.75
	E2	3.75	3.50	3.25
	E3	3.50	3.25	3.00
S2	E1	3.75	3.50	3.50
	E2	3.50	3.25	3.00
	E3	3.25	3.00	2.75
S3	E1	3.50	3.25	3.25
	E2	3.25	3.00	2.75
	E3	3.00	2.75	2.50

The "S", "E" and "INSP" are the "System Behaviour", "Element Behaviour" and "Inspection Level" respectively where:

System Behaviour

System behaviour considers the effect of any existing deterioration and is classified by one of the following categories:

(a) Category S1, where element failure leads to total collapse. This would include failure of main members with no benefit from continuity or multiple load paths, such as a simply supported girder in a 2-girder system;

(b) Category S2, where element failure probably will not lead to total collapse. This would include main load-carrying members in a multigirder system, or continuous main members in bending;

(c) Category S3, where element failure leads to local failure only. This would include deck slabs, stringers, and bearings in compression.

Element Behaviour

Element behaviour considers the effect of any existing deterioration and is classified by one of the following categories:

(a) Category E1, where the element being considered is subject to sudden loss of capacity with little or no warning. This might include failure by buckling; concrete in shear and/or torsion with less than the minimum reinforcement required by Clauses 8.9.2.2 and 8.9.2.3; bond (pullout) failure; suspension cables; eyebars; bearing stiffeners; over-reinforced concrete beams; connections; concrete beam column compression failure; or steel in tension at net section.

(b) Category E2, where the element being considered is subject to sudden failure with little or no warning but would retain post-failure capacity. This might include concrete in shear and/or torsion with at least the minimum reinforcement required by Clauses 8.9.2.2 and 8.9.2.3; steel plates in compression with post-buckling capacity.

(c) Category E3, where the element being considered is subject to gradual failure with warning of probable failure. This might include steel beams in bending or shear; under-reinforced concrete in bending; decks; or steel in tension at gross section.

Inspection Level

Evaluation was not undertaken without inspection. Inspection levels are classified by one of the following levels:

(a) Level INSP1, where a component is not inspectable. This might include hidden members not accessible for inspection such as interior webs of voided slabs;

(b) Level INSP2, where inspection is to the satisfaction of the evaluator, with the results of each inspection recorded and available to the evaluator;

(c) Level INSP3, where inspection of critical and/or substandard components has been carried

8. ANALYSES

The analyses were performed through structural software such as SAP2000 (Ver. 10.) The structure's geometry, material properties, section properties or member sizes, computed loads and the load factors for desired limit states were input. Final factored forces (Moments, Shears, Axial Loads, etc.) including the effects of "DLA" and distribution of live loads were obtained at the critical locations for various components such as girders, deck, cross beams, etc.

9. RESISTANCE CALCULATION

Following the analyses, resistances of various components for flexure, shear, axial loads, etc. were computed in accordance with the relevant sections of CHBDC for different materials. Appropriate "Resistance Adjustment Factors" (U) were applied to the computed resistances. The "U" values were obtained from CHBDC Table 14.13.2 as follows:

Resistance Category	**Resistance Adjustment Factor, *U***
Structural Steel (ϕ per Clause 10.5.7)	
Plastic Moment	1.00
Yield Moment	1.06
Inelastic LTB Moment	1.04
Elastic LTB Moment	0.96
Compression or tension	1.01
Shear (stocky web)	0.87
Shear (tension field)	0.87
Bolts	1.27
Welds	1.32
Rivets	1.81
Composite - Slab on Steel Girder (ϕ per Clauses 8.4.6 and 10.5.7)	
Bending Moment	0.96
Shear Connectors	0.94
Reinforced Concrete (ϕ per Clause 8.4.6)	
Bending Moment	
$p \leq 0.4\, p_b$	1.06
$0.4\, p_b \leq p \leq 0.7\, p_b$	0.99
Axial Compression	1.11
Shear (> min. stirrups)	0.94
Shear (< min. stirrups)	0.82
Prestressed Concrete (ϕ per Clause 8.4.6)	
Bending Moment	
$\omega_p \leq 0.15$	1.01
$0.15 \leq \omega_p \leq 0.30$	0.94

10. CAPACITY/DEMAND (C/D)

The ratios of resistances to the forces for various bridge components were computed (e.g. $\frac{M_r}{M_f}$, $\frac{V_r}{V_f}$ and $\frac{C_r}{C_f}$) at critical locations. The M_r, V_r and C_r values included the resistance adjustment factors "U".

11. POSTING

If all the C/D ratios were equal to or greater than 1.0, no posting was recommended. The values of C/D close to 1.0 were also considered sufficient, if in the evaluation engineer's judgement, there were sufficient factors of safety involved, to ignore the minor deficiency.

Reinforced concrete bridges with C/D ≥ 0.9 were not recommended for posting in accordance with CHBDC 14.17.1. It needs to be mentioned that CHBDC 14.17.1 does not specify the limit of 0.9. It was MRC's opinion to post the reinforced concrete bridges if C/D was significantly below 1.0.

When a posting was required, additional analyses were performed for CL-2-W and CL-3-W loadings and corresponding factored forces were computed. "Live Load Capacity Factor" (F) was computed as follows:

$$F = \frac{U\phi R - \sum \propto_D D - \sum \propto_A A}{\propto_L L(1+\mathrm{I})}$$

where,

$U\phi R$ is the resistance after adjustment,

$\sum \propto_D D$ is the factored Dead Load Force.

$\sum \propto_A A$ is the factored force due to additional loads including wind, creep, shrinkage, temperature and differential settlement.

$\propto_L L(1+\mathrm{I})$ is factored force due to Live Load including the DLA.

The requirement of posting (single or triple) or consideration for closure was checked in accordance with CHBDC 14.7.2 as below:

F > 1	No posting required
1>F≥0.3	For Eval. 1 (CL-1-W) Triple Posting
F<0.3	For Eval. 1 and F>0.3 for Eval. 3 (CL-3-W) Single Posting
F<0.3	For Eval. 3 Consider closure of bridge

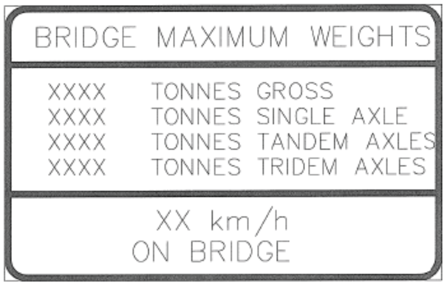
Once the requirement for posting was determined, the axle loads for single, tandem and tridem axle postings were obtained as per CHBDC 14.17.3.3 and regulations set by the Province of Nova Scotia as below:

Gross Weight = 63.7 F

Single Axle = 9.1 F

Tandem Axles = 17.0 F

Tridem Axles = 23.0 F



Sign Size: 8'–0" x 4'–0"

Where,

63.7 is the CL1-W truck weight in tonnes and F is the live load capacity factor for CL1-W loading.

12. CONCLUSIONS

The conclusions were made based on the evaluation results. These would include recommended postings, if required. The recommendations for further inspections and/or testing of specimens might also be included if required as per MRC's opinion or judgement. Recommendation for possible measures which can be adopted in the near future to raise or remove the posting limits might also be made.

13. DISCUSSIONS AND RECOMMENDATIONS

The St. Peter's Canal Bridge is rated as satisfactory for standard CHBDC loading after reducing the "Target Reliability Indices" by 0.25. The application of this provision is a common practice in the Province of Ontario. By adjusting this parameter, MRC recommends that the bridge be inspected at a frequency indicated by the BIM and be re-evaluated within the next five (5) years.

The results of the evaluation indicate that the flexural resistance of the stringers is slightly lower than that required to withstand standard loading (C/D = 0.97). The overall visual inspection indicates no distress in the structure. As the C/D ratio is quite close to 1.0, no posting is being recommended at this time.

APPENDIX
Bridge Evaluation Reports

Public Works and Government Services Canada
Travaux publics et Services gouvernementaux Canada

HIGHWAY BRIDGE INSPECTION FOR PARKS CANADA IN ATLANTIC PROVINCES

St. Peters Canal National Historic Site
Volume 1 of 2
Bridge Inspection Report

MRC
Consultants in Transportation

February 2007

PARKS CANADA

ST. PETERS CANAL

NATIONAL HISTORIC SITE

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VOLUME 1 of 2

BRIDGE INSPECTION REPORT

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EXECUTIVE SUMMARY

McCormick Rankin Corporation was retained by Public Works and Government Services Canada (PWGSC) to undertake an inspection of the St. Peter's Canal Bridge. The inspection was completed on October 3, 2006.

The inspections were completed following the process and procedures set out in the Bridge Inspection Manual 2001 (BIM2001). Visual Inspections were augmented by hammer soundings and probing to assess the current conditions of the various structure components. The inspection was completed as a Comprehensive Inspection. The access for inspection was scaffolding on a barge for the span over the canal, and scaffolding and ladders for the remaining sections.

The bridge is in fair condition with an overall condition rating of 3. The chord members are the main components of concern, as numerous reinforcements were completed to the chord members by welding, a practice generally not desirable on movable bridges. The bottom chords also exhibited medium to severe corrosion particularly at the panel points. Numerous perforations were noted in the lacing bars and the chord flange.

The evaluation of the bridge (under separate cover) indicated the bridge to have adequate structural capacity when evaluated in accordance with the CHBDC. This also accounts for the average section loss noted in the bottom chords.

MRC has recommended that a detailed electrical, mechanical and structural steel inspection, be completed within the next year. The inspection would also include non-destructive testing of fatigue sensitive details. The cost of the inspection is estimated at $20,000.

A detailed inspection of the north bottom chord is also recommended. This inspection would require the removal of the concrete counterweights to facilitate the inspection. The cost of this inspection, including counterweight removal, is estimated at $20,000.

1. INTRODUCTION

McCormick Rankin Corporation was retained by PWGSC to undertake a detailed comprehensive inspection the St. Peter's Canal Bridge for Parks Canada.

The inspection was conducted to the requirements of PWGSC Bridge Inspection Manual 2001. Access was provided by Superport Marine, using scaffold on a barge for the span over the canal. Otherwise access was provided by scaffold and ladders for "hands on" inspection.

Field Inspection was carried out under the supervision of Doug Dixon, P. Eng. The field work was completed under the leadership of Philip Wu, P. Eng. With assistance from Vernu Sivakkolundu, E.I.T.

The field work was completed on October 3, 2007.

This report presents the following:

a) standard bridge inspections forms following the BIM complete with ratings for material condition (MCR) and performance condition (PCR);

b) photographs of the observed condition of the bridge;

c) cost estimates to repair the identified conditions.

The cost estimate is divided into:

a) Immediate Remedial Works for Safety Reasons;

b) Urgent Remedial Works (within two years) ; and

c) Rehabilitation Work within the Next Five (5) Years.

We have further recommended additional Engineering Studies or Surveys (Destructive and Non-Destructive Testing) where MRC has deemed such works appropriate.

Under separate cover (Volume 2) is the result for the Structure Evaluation in accordance with the Canadian Highway Bridge Design Code (CHBDC), CSA S6-00, Chapter 14.

2. BACKGROUND

The St. Peter's Canal Bridge is a steel through truss swing bridge with an open steel grating, except at the end panel of the counterweight span, where a concrete slab was installed. The structural steel floor system consisted of deck grating support beams overlying longitudinal stringers supported on transverse floor beams.

The bridge was constructed circa 1936 and rehabilitated in 1982, 1991 and 1997/1998.

The 1982 rehabilitation included the installation of new decking and steel beam guide rails on the bridge. The decking consisted of concrete slab at the counterweight span (between nodes 2W and 3W), and open steel grating at the remaining sections.

In 1991, the structural steel was cleaned and coated.

The 1997 rehabilitation included the following: patch repairs to the abutments and pier; removed, inspected, and refurbished the pintle assembly; improved approaches by installing steel beam guide rails, approach parapet walls, and traffic loop detectors; and, reinforced various truss diagonals and top chords.

This inspection was a visual structural inspection (augmented with limited non-destructive testing) to assess the condition of the bridge, identify immediate needs and to assist in the identification of future needs and current loading deficiencies.

In general the identification of other deficiencies have not formed part of our assignment. Deficiencies in horizontal and vertical alignment of the approaches or the superelevation or sight distance of the roadway have not been assessed However, MRC has included brief comments on these matters. Functional deficiencies as well as other non-structural issues have generally not been part of our investigations.

3. CONDITION RATING SYSTEM

3.1 General

This section describes the principles and general application of the condition rating system used to assess observed defects in the materials and performance of individual components of a bridge, and the overall or general condition rating for the entire structure as a whole. Also included are guidelines for the application of a priority code for recommended repairs. Observations were augmented with non-destructive test methods such as hammer sounding, chain drags, and the use of ultrasonic gauges to obtain material thickness measurements.

For all concrete components, the surface conditions were observed and recorded. Exposed concrete decks received a chain drag to identify delaminated concrete. Barriers, parapets, sidewalks, piers and abutments were sounded using either normal hammer sounding techniques, chain dragging, or using a Delam® 2000 Rotary Delamination Hammer.

MRC undertook random cover meter readings to assess the depth of concrete cover over the reinforcing steel. This is useful information to use in determining repair quantities and repair strategies. The Team also used concrete crack indicators to assess the width of cracks.

Steel components were inspected as per the BIM. Section loss from corrosion was recorded using callipers or micrometers. So too were the material thickness either side of the corroded area to record "as rolled" thickness. MRC also examined several steel components using an ultrasonic thickness gauge. Significant corrosion pits were recorded. Rivets and bolts were inspected particularly for section loss of rivet heads (from corrosion) or broken connectors due to rust jacking on built up members. MRC reviewed all Class C or worse details.

In the BIM, the material and performance condition rating comprises a numerical system in which a number from 1 to 6, (1 = very severe defect and 6 = new condition) is assigned to each component of the structure based upon the severity of the material defects or the ability of a component to perform its function within the structure. Both material and performance defects perceived are considered from all components. The numerical rating assigned to a particular component(s) reflects the most severe condition of material defects or reduction of performance observed. The component(s) condition rating was assigned without consideration of the importance of the component(s) within the structure.

Components not visible or inaccessible at the time of inspection were noted. The provision necessary for inspection (access, traffic control, etc.) were identified and arrangements made for proper inspection to be carried out.

In addition to the condition rating, each defect is given a summary priority code for remedial action and scheduling. The priority code comprises an alpha character indicative of the urgency and nature of the required repairs to a component or the need for more detailed inspection. Recognition of the importance of the component within the structure was reflected in the assigned priority rating.

The general condition rating of the structure as a whole was based on the most severe component condition rating with some subjective modifications to reflect the importance of the component within the structure; taking also into account the load carrying capacity of the component as determined under the Load Evaluation which is provided under separate cover.

3.2 Bridge Inspection Manual Condition Rating System

The following text is taken from the 2000 inspection reports provided by PWGSC. It has been edited to ensure its relevance to the BIM 2001 Manual and the work as completed by MRC.

3.2.1 Condition Rating for Components of a Structure

Both material and performance defects were considered for all component(s). The numerical rating assigned to a particular component reflected the most severe condition of material defects or reduction of performance.

3.2.2 Material Condition and performance Rating for Components of a Structure

The material condition rating for the components of a structure represent the condition of the component based upon observed defects in the materials of the component.

The application of the material condition rating system to components depends on the type, location and severity of the defects.

The material condition rating represents the worst observed material condition of the component and is based on any one or a combination of the guidelines given under that rating. The inspector recorded the observed material defects and identified the cause producing those defects wherever possible. The inspector takes measurements to quantify the extent and general location of the defects for all components.

The performance condition rating for components of a structure describes the condition of the component based upon its ability to perform its intended function in the structure.

In most cases, the performance defect of a component is closely related to, or attributable to, defects in the component materials as material defects often lead to performance defects. The severity of the performance defect is not necessarily the same as the severity of the material defect. The performance condition rating was assigned on the basis of the approximate capacity of the component to perform its intended function within the structure and the classification of the component, i.e. primary secondary or auxiliary.

In some cases, performance defects exist due to defects in design or construction and may not be directly related to material defects. Also, performance defects in a component may be the result of unexpected behaviour of the structure or due to performance defects in other components of the structure. The inspector recorded the observed reduction in performance and the causes producing those effects wherever possible.

3.3 Code for Priority of Component Repair of a Structure

The priority code assigned to each component is one of the following:

- 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 reassessed at the next inspection.

All components were assigned a priority code indicative of the urgency and nature of recommended repairs or need for further inspection. Performance related deficiencies were considered to be of higher priority than material related defects. Nevertheless, the objectives of the recommended rehabilitation program was to address, where possible all material and performance related defects.

Recognition of the importance of the component within the structure was reflected in the priority rating assigned. Recognition of the importance of the component was achieved by the classification of all components as either primary, secondary or auxiliary. The classification is generally along traditional structural behaviour except for non-structural components.

When the component condition rating indicated a significant level of deterioration or loss of performance, yet the recommended repairs are assigned a low priority, a brief written explanation is provided noting the component classification and nature of the deficiency.

3.3.1 Numeric Condition Rating of Structure

The general condition rating of a structure is an indicator of the most severe material or performance defects of a primary component or a modified indicator of the most severe material or performance defects of a secondary or auxiliary component, with some subjective modification to reflect the importance of the component within the structure and its load carrying capacity from the results of the Load Evaluation Rating.

The general condition rating of the structure consists of the lowest number from 1 to 6 obtained from the condition rating for the components of the structure as follows:

a) The lowest rating of a primary component;

b) The lowest rating of a secondary component plus one;

c) The lowest condition rating of an auxiliary component plus two (not to be less than 4).

The addition of "plus one' in b), and "plus two" in c), is to reflect the somewhat lesser importance of the secondary and auxiliary component(s) relative to the primary component(s) rating.

4. DISCUSSION AND RECOMMENDATIONS

The bridge is in fair condition with an overall condition rating of 3.

The bottom chords are the main components of concern. The bottom chords exhibited medium to severe corrosion particularly at the panel points. Numerous perforations were noted in the lacing bars and the chord flange. We also noted that numerous reinforcements were completed to the chord members by welding. Moveable bridges are subject to load reversals, resulting in stress reversals in many members at some point during the movement of the bridge. Welded repair/reinforcement is generally not desirable in the tension zone, due to fatigue issues, and particularly to those members subject to stress reversal. In addition, welding results in shrinkage and residual stresses in the reinforcement.

We would recommend that a detailed structural steel inspection, including non-destructive testing, be completed within the next year. The work should also include the inspection of the electrical and mechanical components. The cost of the inspection is estimated at $20,000. Since the concrete counterweights were installed on the north bottom chord, we would recommend that a detailed inspection of the north bottom chord be completed. This would involve the removal of the concrete counterweights to facilitate the inspection. The cost of that inspection, including the removal of the counter weights, is estimated at $20,000. We have included a sample Non-Destructive Testing Plan in Appendix E.

The evaluation of the bridge (under separate cover) indicated the bridge to have adequate structural capacity when evaluated in accordance with the CHBDC. The capacity to demand ratio for stringers is nominally below 1.00 (C/D = 0.97), therefore we do not recommend the bridge be posted. The evaluation of the bottom chords indicated a capacity to demand ratio of 1.24. Therefore accounting for an average section loss of 15% to 20% as observed in the field, the bottom chords have adequate structural capacity.

We have recommended reinforcements to the bottom chords at the panel points due to medium to severe corrosion with localized perforations at the chord flange and lacing bars. The extent of the reinforcements should be based on further structural steel inspection. The work is estimated at $780,000 including engineering and contract administration.

The coating is also approaching the end of its life. We would recommend that the structural steel be cleaned and coated after all necessary repairs and reinforcement to the structural steel are completed. We have not provided a cost estimate to complete the cleaning and coating, as it is anticipated that the work is beyond the five (5) year time frame.

At both approaches to the bridge, the horizontal alignment is generally poor. A crest vertical curve is also noted at the west approach to the bridge. The poor alignment combined with obstructions from vegetation resulted in limited sight distance of the traffic signals and various warning signs.

The steel beam guide rail at the west approach was in good condition. The height of the steel beam guide rail was measured to be 600mm. The various transition and end treatment details are not in accordance with the standards. An independent memo is appended to the end of this report.

APPENDIX A
Bridge Inspection Reports