PARSONS

LaSalle Causeway Bascule Bridge

2018 Fatigue Inspection and Evaluation Report

March 1st 2019

delivering a better world

Table of Contents

List of Tables

List of Figures

List of Appendices

- Appendix A – Fatigue Inspection Tables – Main Members
- Appendix B – Fatigue Inspection Tables – Stringers and Other Members
- Appendix C – Brouco NDT Report
- Appendix D – Mequaltech Pins Phased Array Testing Report
- Appendix E – Original 1914 Drawing, Sheet 09

Executive Summary

Parsons was retained by Public Services and Procurement Canada (PSPC) in August 2018 to perform a fatigue inspection of all members and a fatigue evaluation of the primary truss members of the LaSalle Causeway Bascule Bridge (Bascule Bridge). This assignment has included the collection and review of all relevant available data; identification of fatigue prone details, preparation of a fatigue inspection plan, completion of a field condition survey, including non-destructive testing (NDT), structural calculations to estimate the residual Fatigue Life of primary truss members; providing a report summarizing the findings, calculations results, recommendations and any repairs deemed necessary.

The Bascule Bridge is a single leaf Strauss heel trunnion bascule bridge, designed by The Strauss Bascule Bridge Co. of Chicago, and constructed in 1916. The main span of the bridge consists of a modified Warren through-truss with a span length of 48.77 m (160'). The center-to-center truss width is 8.23 m (27') and the center of bottom chord to center of top chord height varies from the east end to the west end from 6.10 m (20') to 7.92 m (26'). The concrete counterweight weighs approximately 500 tonnes (1,100,000 lbs).

In addition to being an important road link, the Bascule Bridge also provides marine access to the inner harbor of Kingston, lifting an average of 900 times per year, and access to the southern entrance of the Rideau Canal.

CAN/CSA-S6-14 The Canadian Highway Bridge Design Code (CHBDC) includes both standard fatigue loading (highway live load) and special load cases for bascule moving bridges. The fatigue evaluation of the Bascule Bridge has been performed by Parsons as per Sections 3, 10, 13, and 14 of the CHBDC, for vehicular traffic on the closed bridge and for bridge opening load cases. The residual life calculations were performed in accordance with the AASHTO Manual for Bridge Evaluation (3rd ed.)

Fatigue Inspection

During the preparation of the fatigue inspection plan, fatigue prone details such as net section of rivetted connection, beam coped ends and welds were identified from existing plans and previous reports. Load induced fatigue prone details of Category D, E or E1 were all targeted for inspection. Distortion induced fatigue details such as web gaps at floorbeam stiffeners were also targeted to be inspected, as well as potential constraint induced fatigue details at floorbeam ends (3 weld intersection with potential triaxial load state). The fatigue inspection has revealed more than 12 cracks in main members and 39 cracks at coped ends of stringers, of which only 16 were noted in previous Comprehensive Detailed Inspections (CDI). The phased array ultrasonic testing (PAUT) revealed no cracks in the main pins of the trunnions and counterweight links, but anomalies indicating potential surface corrosion was found on both counterweight trunnion pins at nodes 21S and 21N.

Due to the structure's configuration, some areas were not accessible for inspection. The most critical are the end of member 13-22 at node 22 and the steel frame of the counterweight.

Some of the cracks found in main members and stringers are more concerning because of their length, orientation, location or because of the occurrence of systematic cracking in several adjacent members. Table E- 1 lists the significant findings of the fatigue inspection and recommendations, as well as associated BIM Priority Codes.

Table E- 1: Summary of Fatigue Inspection Recommendations

Member	Defect	Recommendation	BIM Priority Code	Traffic Impact
Stringers	Cracks at cope ends	*Perform systematic Magnetic Particles testing of all stringers coped ends*	A	None
		Repair all cracks (drill crack tip and install tensioned ASTM A325 bolts or create radius as described in Chapter 5 of FHWA Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges).	A	None
		Monitor the location monthly before repairs and yearly after repairs, to assess the efficiency of repair	A	None
Counterweight Trunnion Pins (21S and 21N)	*Surface anomalies indicating potential surface corrosion*	*Perform Phased Array Ultrasonic Testing (PAUT) to monitor progression of potential corrosion*	C	30 minutes closures
		Investigate corrosion mitigation options (penetrant rust inhibitor or other)	B	n/a
Bottom chord gusset 16N	*Possible cracking on inside gusset plate*	*Grind smooth the suspected crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	B	None
Main truss bottom chord 14S-15S	*Multiple cracks originating from perforation in cover/batten plate near 15S*	*Grind smooth the crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	C	None
Main truss bottom chord 14S-16S	*12mm crack originating from perforation in web of inside Channel*	*Grind smooth the crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	A	None
Bottom chord Gusset 16S	*3mm crack, perpendicular to main stress on inside gusset plate, originating from a perforation in the plate.*	*Grind smooth the suspected crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	C	None
Main truss top chord 3S-5S	*60mm lamination with potential crack, mostly horizontal and parallel to main stress, in channel. 25mm area with multiple laminations or probable cracks, diagonal to main member stress.*	*Monitor defect monthly before repair and yearly after repair*	*A*	*None or 30 minutes closures*
		Reinforce affected channel with slip critical bolted steel element	*A*	*1 Lane closure*
Main truss vertical 1N-2N	*3mm crack, perpendicular to main stress, in flame cut hole*	*Grind smooth the suspected crack or drill crack tip and install tensioned ASTM A325 bolt*	*A*	*1 Lane closure*
		Monitor the location yearly to assess the efficiency of crack grinding	*D*	*None or 30 minutes closures*
Main truss vertical 2N gusset	*Full width horizontal hairline crack and multiple laminations in interior gusset*	*Perform refined FEM analysis of the node to assess the need for strengthening. Assume inside gusset cracked at 100% in analysis*	*A*	*n/a*
		Repair gusset by plating, if required	*A*	*t.b.d*
Vertical 9S-10S	*3mm crack, parallel to main stress, found in the Heat Affected Zone (HAZ) of a removed weld (angle holding sign).*	*Remove angles holding sign to gain complete access to HAZ. Grind welds, HAZ and crack smooth.*	*B*	*30 minutes closures*
		Perform Magnetic Particles testing of the HAZ zone, to check for cracking. Reinstate appropriate coating.	*B*	*30 minutes closures*
		Monitor the location yearly to assess the efficiency of crack grinding	*A*	*None or 30 minutes closures*
Counterweight truss bottom chord 21N-27N	*Full width 100mm cracking of the horizontal flange of bottom interior angle. Crack is perpendicular to main stress. Appears to be through thickness.*	*Reinforce affected angle with slip critical bolted steel element*	*A*	*1 Lane closure*
		Monitor defect monthly (if not repaired before May 2019)	*A*	*None*
Main truss diagonal 13N-16N	*3mm crack found at impact damage, removed by grinding*	*Monitor the location yearly to assess the efficiency of crack grinding*	*D*	*30 minutes closures*

Table E- 1: Summary of Fatigue Inspection Recommendations (continued)

Member	Defect	Recommendation	BIM Priority Code	Traffic Impact
Tower truss horizontal member 15S-18S	*3mm horizontal indication at the edge of channel's web perforation, possibly a crack initiation*	*Grind smooth the suspected crack and the perimeter of the perforation.*	*A*	*None*
		Monitor the location yearly to assess the efficiency of crack grinding	*D*	*None*
		Reinforce corroded element with slip critical bolted steel element	*B*	*None*
Tower truss horizontal member 18S-19S	*60mm vertical perforation with crack in the remaining material, perpendicular to main stress, at node 18S*	*Grind smooth the suspected crack and the perimeter of the perforation.*	*A*	*None*
		Monitor the location yearly to assess the efficiency of crack grinding	*D*	*None*
		Reinforce corroded element with slip critical bolted steel element	*A*	*None*
Verticals 1-2, 5-6, 9-10, and 13-14, North and South	*Crack in 13N-14N, discovered in 2016, reinforced in 2017*	*Reinforce the verticals 1-2, 5-6, 9-10 North and South to create continuity in web plate at lower end of member*	*C*	*1 Lane closure*
All primary truss members	*Tack welds*	*Grind welds smooth. Perform Magnetic Particles testing of the HAZ zone,*	*C*	*1 Lane closure*
		Reinstate appropriate coating	*C*	*1 Lane closure*
All primary truss members	*Debris, spider webs, scale, etc.*	*Coordinate bridge cleaning and bridge inspection to have the best visual possible on all elements and ease crack finding.*	*M*	*30 minutes closures*
All floorbeams end plates	*Potential constraint induced fatigue details*	*Monitor location at each fatigue inspection*	*D*	*None*
		Retrofit the potential constraint induced fatigue details of the end plates by weld grinding	*C*	*None*
All members	*Location with known cracks, repaired or not*	*Monitor location at each fatigue inspection*	*D*	*1 Lane closure*
Main members, stringers and floorbeams	*Unknown steel toughness*	*Perform Charpy testing on steel coupons*	*B*	*None*
Main members, stringers and floorbeams	*Fatigue Prone details*	*Perform Biannual Fatigue Inspections*	*n/a*	*1 Lane closure*

Fatigue Evaluation

The fatigue evaluation of bridge primary truss members for vehicle live loads as per AASHTO MBE methodology shows that few members are close to reaching the end of their theoretical fatigue life under the most conservative assumptions. All members have fatigue residual life of more than 24 years for less conservative but plausible assumptions. In summary:

- No member has reached its theoretical Category C or D fatigue life for any level of fatigue life (minimum, Evaluation 1, Evaluation 2 or Mean) and for any of the traffic growth rate scenario considered (g=0.83% or g=2%)
- Four members are within 15 years of reaching the theoretical end of their Category D fatigue life for the conservative minimum level (equivalent to design life), when the 0.83% traffic growth rate scenario is considered. This scenario gives the lower bound fatigue life. These members include:
  - 4S-6S, 6S-8S, 4N-6N and 6N-8N
- All members have a Category C or D fatigue residual life of more than 24 years for any of the traffic growth rate scenarios considered (g=0.83% or g=2%) for fatigue live level Evaluation 1, Evaluation 2 or Mean.

The fatigue evaluation for the bridge opening cycle adopted a Lower and Upper bound approach. Lower and Upper bound scenarios are defined by different hypothesis on opening parameters such as maximum operating angle, impact force, etc. Results show that the most vulnerable members are located in the counterweight truss. Members with the shortest residual life are 25N-26N, 25S-26S, 21N-27N, 21S-27S and 26S-27S, 26N-27N (with conservative assumptions for net section). Those members would have reached their theoretical fatigue life if detail Category D is considered, for both upper and lower bound scenarios of residual life. Since those members have internal redundancy, the rationale of AASHTO MBE sections 7.2.1 and C7.2.1 to use Category C is applicable. With Category C and for the upper bound of residual fatigue life, these members would have a residual life in excess of 150 years. However, they would have reached the end of the theoretical fatigue life 16 to 37 years ago in the Lower bound scenario for residual fatigue life.

Most of the cracks found in the structural members are in corroded or perforated areas. Given the fact that corrosion and perforations have developed during the service life of the bridge and that only one crack has been found at the net section of a rivetted connection (13N-14N, 2016), it is possible to assume that sharp indentations in corroded or perforated areas have a fatigue behavior worse than a Category D fatigue detail. Corrosion and perforations allow cracks to initiate and propagate under less loading cycles. Sharp indentations created by corrosion should be grinded smooth. Perimeter of perforated areas should also be ground smooth and reinforced locally on members most vulnerable to fatigue. Limiting further section loss is critical to prevent an increase in stress range that would lead to a reduction of fatigue residual life. As some members reach the end of their theoretical fatigue life for Category D or even for Category C, cracking at net section of riveted connections might appear in members with the highest stress ranges. Regular fatigue inspection targeting the most vulnerable members is recommended to mitigate the risk associated with fatigue cracking. Recommendations to prevent crack initiation and for crack detection are presented in Table E- 2.

Table E- 2: Summary of Fatigue Evaluation Recommendations

Fatigue Evaluation Recommendations
Prevent progression of corrosion in steel members, in particular, members with high stress range from vehicular traffic or from bridge opening.
Perform biannual fatigue inspections targeting the most fatigue vulnerable members to allow early detection of fatigue cracks. Special attention should be given to net section of riveted connections and at sharp indentations in corroded of perforated areas.
Perform preventive maintenance to remove sharp indentations created by corrosion and repair perforations. Grind sharp indentations and perimeter of perforation. Add slip critical bolted plates when necessary. Priority should be given to the most fatigue vulnerable members.
Maintain bridge coating system in good condition to prevent further section loss and associated increase of fatigue stress range
Maintain bridge coating system in good condition for early crack detection.
Coordinate inspections and bridge cleaning for early crack detection

1.0 Introduction

The LaSalle Causeway (the Causeway), owned and operated by Public Services and Procurement Canada (PSPC), carries County Road No. 2 across the Cataraqui River within the City of Kingston, providing an important transportation link between the downtown area on the west side of the river with the Barriefield/CFB Kingston area on the east side of the river. Approximately 25,000 vehicles cross the Causeway daily, with approximately 2% commercial vehicles. The Causeway consists of five (5) interconnecting structures: the west bridge (including its west approach), the west wharf, the bascule bridge, the east wharf, and the east bridge (including its east approach). The bascule bridge also provides marine access to the inner harbor of Kingston, lifting an average of 900 times per year, and access to the southern entrance of the Rideau Canal. Average opening per year have varied in the life of the structure. For some years, the number of openings is unknown. It is estimated that the bridge has opened approximately 193 000 times since its construction was completed on April 15, 1917. The location of the Causeway is shown on the key plan in Figure 1.

Parsons was retained by PSPC in August 2018 to perform a fatigue inspection of all members and a fatigue evaluation of the primary truss members of the LaSalle Causeway Bascule Bridge (the Bascule Bridge). The assignment included the collection and review of all relevant available data; identification of fatigue prone details, preparation of a fatigue inspection plan, completion of a hands-on inspection, including non-destructive testing (NDT), structural calculations to estimate the residual fatigue life of primary truss members; providing a report summarizing the findings, calculations results, recommendations and any repairs deemed necessary.

This report documents and summarizes the findings of the fatigue inspection and fatigue evaluation of the Bascule Bridge and includes recommended repairs for the structure.

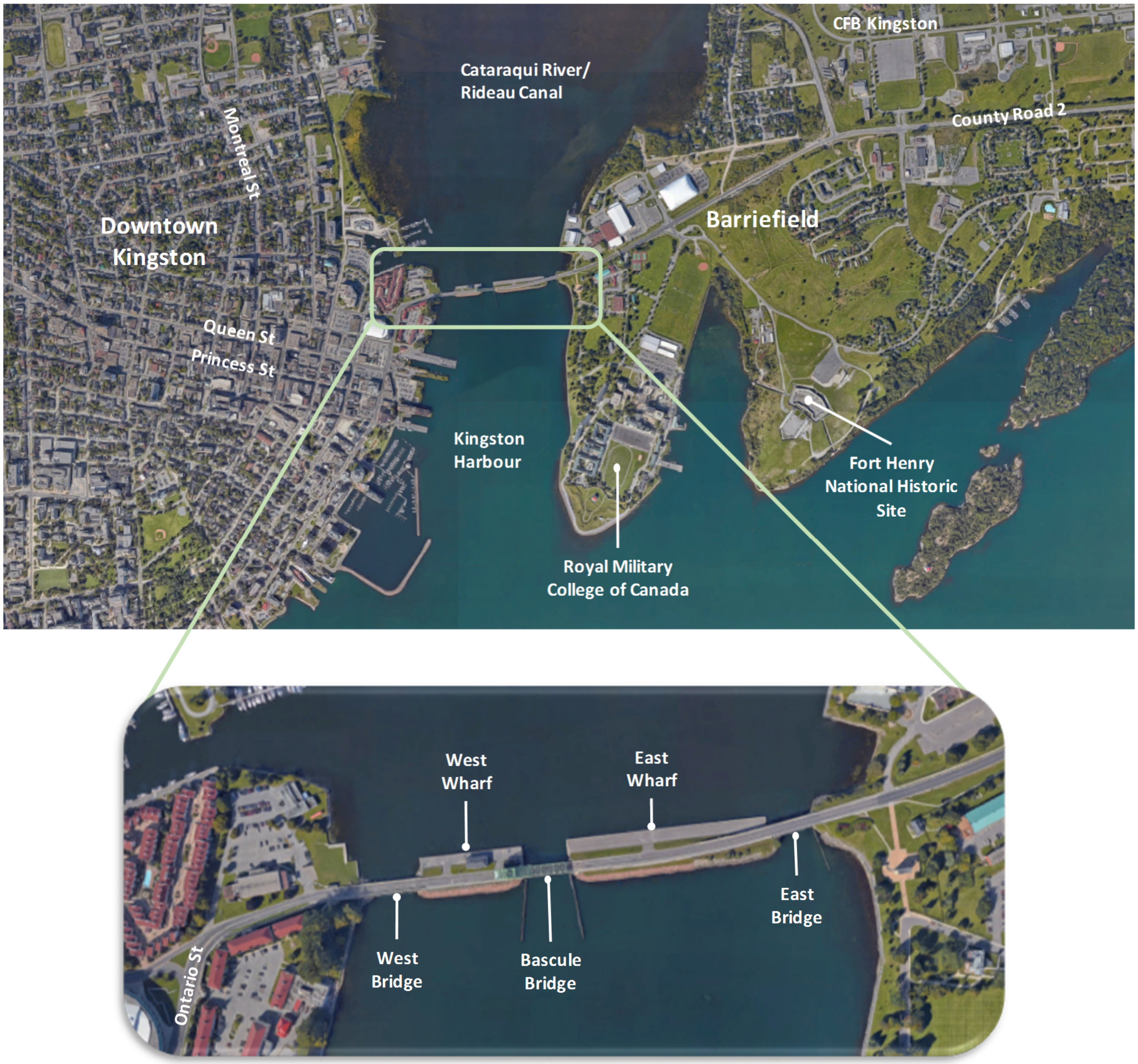


Figure 1: Key Plan

2.0 Structure Description

The Bascule Bridge is a single leaf Strauss heel trunnion bascule bridge, designed by The Strauss Bascule Bridge Co. of Chicago, and constructed in 1916. The main span of the bridge consists of a modified Warren through-truss with a span length of 48.77 m (160’). The center-to-center truss width is 8.23 m (27’) and the center of bottom chord to center of top chord height varies from the east end to the west end from 6.10 m (20’) to 7.92 m (26’). The concrete counterweight weighs approximately 500 tonnes (1,100,000 lbs.).

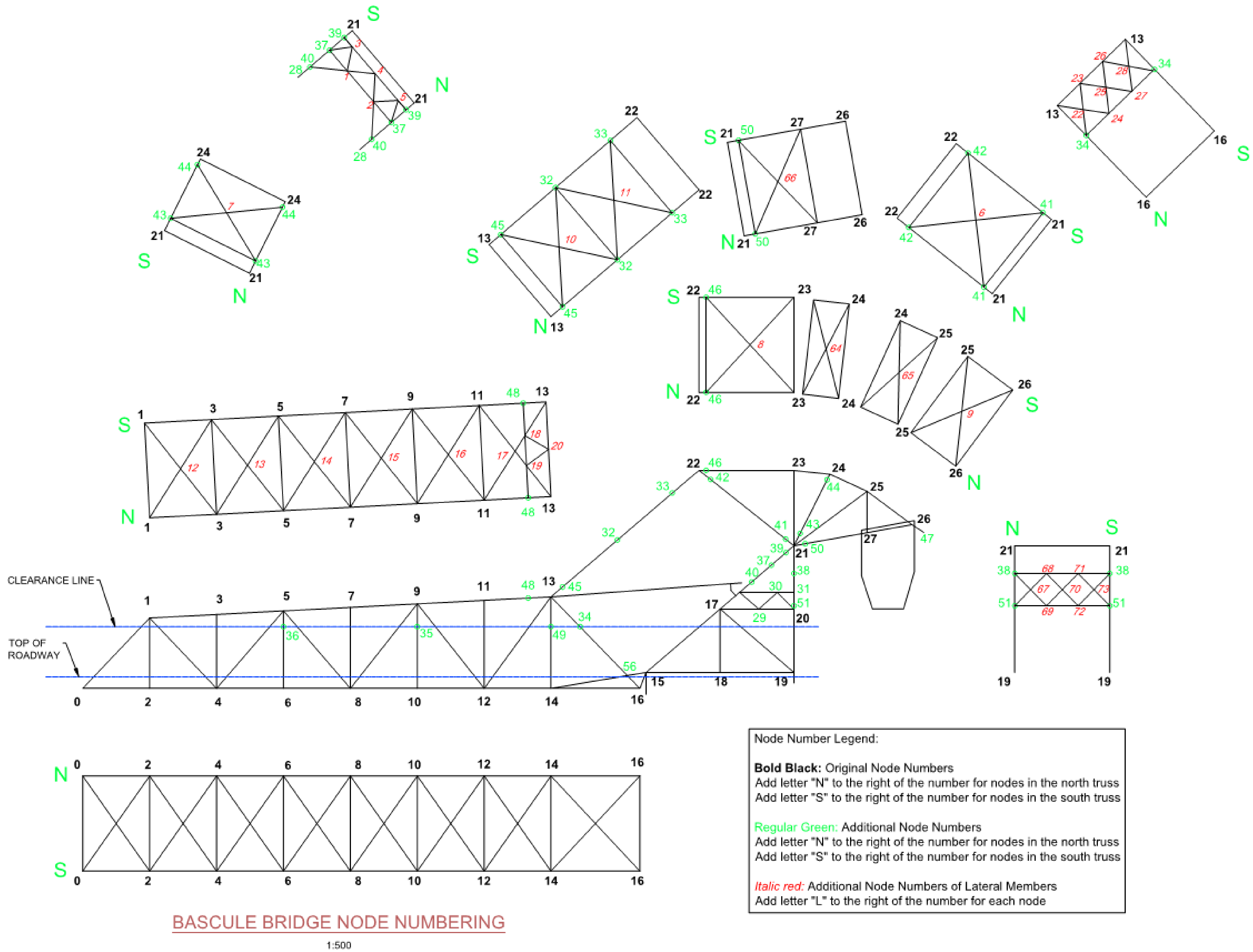
The deck is made of an open steel grating, supported by sills, stringers and floorbeams. This existing deck was installed in 1972-1973 to replace the original deck. Original stringers were spaced to accommodate railroad or street car tracks, as shown on sheet 09 of the original drawings (see Appendix E). No drawings showing the installation of tracks were found, and there is no historical documentation of trains or street cars on the Causeway based on research by PSPC. The original spacing of stringers was maintained when the original stringers were replaced in 1972-1973.

2.1 Fatigue Prone Details

Existing drawings, reports, and other documentation were reviewed to identify all load-induced (Category D or more severe), distortion-induced, or constraint induced fatigue-prone details as per the CHBDC fatigue detail classification and the FHWA Inspection of Fracture Critical Bridge Members manual. Cope beams fatigue detail category varies from one reference to the other. No category for that detail is given in CHBDC, while it is an E1 detail as per CAN/CSA S16-14 Design of Steel Structures and a category C detail as per AASHTO LRFD (6^{th} Ed.) table 6.6.1.2.3-1. The CISC FAQs were consulted to clarify the fatigue detail category associated with coped beams. Based on that reference, coped beams are considered as class E1 for inspection purposes. Appendix A presents the list of locations targeted for inspection in the fatigue inspection plan, along with the type of inspection or testing required.

2.2 Nomenclature

For reference within this report, primary members of the Bascule Bridge, truss nodes and member numbers were adopted from 1915 original drawings. For some secondary members, no node numbers are provided on the original drawings. To address these nodes in this evaluation report, additional node numbers were defined. Node numbering was graphically presented in Figure 2 and in more detail on SK-01 in Appendix A of the 2017 Structural Evaluation Report by Parsons.



Node Number Legend:

Bold Black: Original Node Numbers
Add letter "N" to the right of the number for nodes in the north truss
Add letter "S" to the right of the number for nodes in the south truss

Regular Green: Additional Node Numbers
Add letter "N" to the right of the number for nodes in the north truss
Add letter "S" to the right of the number for nodes in the south truss

Italic red: Additional Node Numbers of Lateral Members
Add letter "L" to the right of the number for each node

Figure 2: LaSalle Causeway Bascule Bridge Node Numbering

3.0 Historical Information

3.1 Maintenance and Inspection History

A detailed maintenance and inspection history of the Bascule Bridge has been incorporated into the LaSalle Causeway 2016 Annual Comprehensive Detailed Inspection Report. It is summarized here as it outlines members which have been replaced or strengthened and forms part of the basis upon which the 2017 Structural Evaluation (under a separate cover) and the 2018 Fatigue Evaluation were conducted.

3.1.1 MAINTENANCE HISTORY

The Bascule Bridge has undergone numerous repairs and rehabilitation works since its construction in 1916. The following is list of some of the major works undertaken:

- In 1966, the Bascule Bridge operating mechanism was renewed, including a new operator's cabin and control electronics.
- In 1972-1973, the Bascule Bridge floor beams were replaced with welded wide flange beams, the floor system stringers were replaced, along with the timber sidewalk stringers and deck planks.
- In 2001, two buffers were added to the Bascule Bridge to reduce impact when the bridge lands.
- In 2004, a maintenance contract on the Bascule Bridge was completed to replace rivets in fatigue prone members of the counterweight.
- In 2009-2013, the Bascule Bridge underwent a major rehabilitation, which included: removal of lead paint and application of low VOC protective coating system to all structural steel members of the bridge; structural repairs to deteriorating steel members (i.e. operating arm, bottom gusset plates, exterior splice plates in the bottom chord, etc.); reinforcing certain diagonal truss members; replacing timber sidewalk stringers and deck planks; installation of new pedestrian railing meeting CHBDC requirements for a combination pedestrian/bicycle barrier; replacing traffic barrier; and replacing wood stairway treads with steel treads; installation of a new steel counterweight for balancing the bridge.
- In 2016, new W-beam guide rails were installed on the northeast and northwest approaches of the west bridge, along the south side of the sidewalk on the west wharf, along the south side of the sidewalk on the east wharf and along the northeast and southeast approaches of the east bridge. New pedestrian railings were installed on the southeast and southwest wingwalls of the Bascule Bridge, and repairs were carried to the southeast and southwest training walls. New chain link fences were installed on the southeast and southwest embankments of the Bascule Bridge and at the westbound traffic barrier.
- In 2017, the following rehabilitation and inspection contracts for the Bascule Bridge were initiated: replacement of the buffers; detailed inspection and repairs options report for the steel deck grating; repairs to the span locks and bottom chords of the leaf truss; upgrades to the motor drive and motor control upgrade, rehabilitation of the guide assemblies, reinforcement of members 13N-14N and 13S-14S following the discovery of cracks in interior angles of 13N-14N.

The list of repairs and rehabilitation works above is not comprehensive, as not all information was available to the inspectors. However, it should highlight the major works undertaken to the structures on the Causeway.

3.1.2 INSPECTION HISTORY

Parsons performed the following recent inspections on the Bascule Bridge:

- 2018 - LaSalle Causeway 2018 Annual Comprehensive Detailed Inspection and Fatigue Inspection; Project Number: R. 0090045.001;
- 2017 - LaSalle Causeway Deck Grating Inspection and Repair Report; PWGSC Project No. R.082857.001;

- 2016 - LaSalle Causeway 2016 Annual Comprehensive Detailed Inspection Report; Project Number: R. 055058.002;
- 2015 - LaSalle Causeway Bascule Bridge 2015 Detailed Measurements; Project Number: R. 055058.002;
- 2015 - LaSalle Causeway 2015 Annual Comprehensive Detailed Inspection Report Project Number: R. 055058.002; and
- 2015 – Document Review Report. Project Number R.055058.002.

3.1.3 BRIDGE SPECIFIC REFERENCE MATERIAL

The following relevant reference material has been reviewed for this evaluation:

- 2017 – Parsons Inc – LaSalle Causeway Bascule Bridge Structural Evaluation Report, PSPC Project No. R055058.002;
- 2017 - Parsons Inc - LaSalle Causeway Deck Grating Inspection and Repair Report, PWGSC Project No. R.082857.001;
- "LaSalle Causeway – Bascule Bridge, Replacement of Span Locks"; Issued for Tender Drawings; Project No. R.082857.001; Parsons Inc.; November 2016;
- 2016 – Parsons Inc - LaSalle Causeway 2016 Annual Comprehensive Detailed (Bascule) & General Inspection Report, PWGSC Project No. R.055058.002;
- "LaSalle Causeway 2015 Annual Comprehensive Detailed Inspection Report"; Report; Parsons; September 2015;
- "LASALLE CAUSEWAY Trunnion Joint Inspection and Analysis Report"; Report; MMM Group; June 5, 2015;
- "LaSalle Causeway Comprehensive Detailed Inspection"; Genivar, March 2014;
- "2011 Comprehensive Detailed Inspection Report for LaSalle Causeway"; Delcan Corporation; December 2011;
- "2010 Comprehensive Detailed Inspection Report for LaSalle Causeway"; McCormick Rankin Corporation; March 15, 2011;
- "LaSalle Causeway – Bascule Bridge, Repairs and New Coating"; As-Built Drawings S01 to S25; Project No. R.012359.001; McCormick Rankin Corporation; September 16, 2010;
- 2005 - McCormick Rankin Corporation - Kingston Bascule Bridge Fatigue Review and Rehabilitation of Counterweight Members, January 2005;
- "Kingston Bascule Bridge – Fatigue Review and Rehabilitation of Counterweight Members (Updated After Construction)"; McCormick Rankin Corporation; January 2005;
- 2001 - McCormick Rankin Corporation – Seismic Structural Analysis of the LaSalle Bascule Bridge, October 2001;
- 1998- Fatigue Probabilistic Assessment of the LaSalle Causeway Bascule Bridge, prepared by the Technology Directorate, Architectural and Engineering services, Public Work & Government Services Canada;
- 1997 – LaSalle Causeway Bascule Bridge Fatigue Investigation Report, prepared by David C. Stringer Engineering Inc.;
- 1973 - C.C. Parker and Associates Limited - LaSalle Causeway – Repairs to Bridges As-Built Drawings, PWGSC Project Number 81254, prepared by C.C. Parker and Associates Limited, March 1973;
- 1971 - "Bascule Bridge – Repairs to Floor System": Design Drawings: Sheet 1 of 1, Public Works of Canada, Ontario Region, December 1971; and
- 1915 - "Strauss Trunnion Bascule Bridge (Patented) over Cataraqui River, Kingston Harbor Improvements for Dept. of Public Works"; As-Built Drawings (1 to 22); The Strauss Bascule Bridge Co., Chicago; January 21, 1915.

4.0 Fatigue Inspection

4.1 General

This section summarizes the methodology of the fatigue inspection and the observed condition of the structural members, i.e. main truss members, floorbeams and stringers. Observations are recorded in fatigue inspection tables in Appendix A for the main members, and Appendix B for stringers and other members. The Brouco NDT report is included in Appendix C, and the Mequaltech Pins Phased Array testing report is included in Appendix D.

The scope of the fatigue inspection included the following:

- Identification of fatigue prone details in floorbeams, stringers and main truss members subjected to structural fatigue;
- Preparation of an inspection plan, including non-destructive testing (NDT) inspection according to the terms of reference;
- Visual inspection of all members/elements listed in the plan; and
- Perform NDT at locations targeted in the inspection plan and at locations identified as suspect during the visual inspection. NDT included magnetic particle testing (MT), ultrasonic testing (UTT) and phased array ultrasonic testing (PAUT) for trunnions and counterweight link pins.

4.2 Methodology

The fatigue inspection was planned to target fatigue prone details on floorbeams, stringers and main truss members susceptible to fatigue (as defined by CHBDC section 10.17.2.1 for definition). Load induced fatigue, distortion induced fatigue, corrosion induced fatigue and constraint induced fatigue were considered. The 2017 structural evaluation was used to determine which members are subjected to fatigue from vehicular live loads or from the bridge opening cycle. An inspection plan was prepared, according to the Terms of Refence.

The inspection team included a senior bridge engineer, and two senior NDT technicians, one of which specializes in PAUT testing. To facilitate access for a hands-on visual inspection by the inspection team, trained access personnel and specialized access equipment were utilized as required (e.g. Level 3 Rope Access Technician or Marine Access Specialist). Suspect areas were identified during the visual inspection and targeted for NDT testing under the supervision of the bridge engineer. Defects were documented by the NDT technicians and are presented in the specialized reports (Appendixes C and D). Where possible, small defects were removed by grinding, under the supervision of the bridge engineer.

Although a thorough hands-on inspection of the bridge was performed, the possibility that some existing cracks went undetected cannot be ruled out. The nature of the rivetted structure, the configuration of joints, the presence of debris on the structure and the extend of deterioration of the steel components created many potential crack initiation locations in areas that were difficult to inspect, such as truss nodes. Additionally, some areas were inaccessible for inspection, such as the end of member 13-22 at node 22 and the steel frame of the counterweight.

4.3 Deck Components

4.3.1 STRINGERS

The original stringers were replaced in 1972-1973 with new rolled steel sections with coped ends. Notches were created in the webs of the stringers at the coped ends. Previous inspections noted many locations with either cracked paint, crack like indication[1] in steel or cracks in steel. All coped ends of stringers were subject to a hands-on visual inspection, and MT testing at all locations where cracks in paint, crack like indication[1] or cracks in steel were previously documented. Any new locations of cracking at coped ends and random coped ends (20% that were visually inspected) were also tested. The connecting angles radius and bolt holes periphery where also all visually inspected and some locations were subjected to MT testing. At a few locations, triangular stiffeners were found to be welded to stringers, creating fatigue prone details (load and constraint induced fatigue). These locations were all visually inspected, and some locations were subjected to MT testing.

4.3.2 FLOORBEAMS

Original floorbeams were replaced in 1972-1973 with new welded wide flange (WWF) steel sections made of CSA G40.12 steel. The 1972-1973 stringers are connected to the floorbeam with clip angles having approximately the same height as the stringers. At some locations, stringers are connected using angles spanning the full height of the floorbeam webs and ground to bear on the bottom flange. At the connection with stringers, distortion induced fatigue is possible in the web of the floorbeams (see Figure 3: Potential Distortion-Induced Fatigue Detail). However, the probability of web cracking in floorbeams at this location is relatively low because the connections are bolted instead of welded. However, since previous CDI detected web distortions at those connections, it was decided to treat them as potential distortion induced fatigue detail. All such locations were subject to a hands-on visual inspection and some locations were subject to MT testing.

Figure 3: Potential Distortion-Induced Fatigue Detail

Floorbeams are connected to the main trusses by a moment rigid connection. An end plate is welded to the web and flanges of the floorbeam and bolted to the main trusses. At the web, flange and base plate joint, the WWF flange-web welds intersect with the base plate welds, creating an intersection of 3 welds (see figure 108 of Appendix C and Figure 4: Potential Constraint-Induced Fatigue Detail of this report). This detail could be constraining the web into a tri-axial state of stress (tension in the truss bottom chord, tension in the floorbeam flange, tension in half of web height and shear load in the web). All such locations were subject to hands-on

[1] Crack like indications are defects for which previous inspections couldn't statute if they were indeed a crack or if they were something else, like a linear perforation caused by corrosion, a notch made by tools or something analog.

visual, and some were tested with MT. The welded splice in floorbeam 16 was also tested with MT (see figure 107 of Appendix C).

Figure 4: Potential Constraint-Induced Fatigue Detail

4.4 Superstructure

All main truss members undergoing stress reversal with the bridge opening operation and/or under traffic loading as per CHBDC clause 10.17.2.1 were identified for fatigue investigation. Data from the 2017 structural evaluation was used for this purpose. Previous inspection reports, original plans and previous reports were reviewed to identify all fatigue prone details of targeted members. Appendix A presents the list of main truss members and locations that were inspected.

4.5 Significant Findings and Recommendations

The fatigue inspection revealed more than 12 cracks in main members and 39 cracks at coped ends of stringers, of which only 16 were noted in previous inspections. The PAUT testing revealed no cracks in the main pins of the trunnions and counterweight links, but anomalies indicating potential surface corrosion were found on both counterweight trunnion pins at nodes 21S and 21N. Appendix A to D present exhaustive details of defect type and location.

Tables in Appendix A present probable causes of crack initiation based upon inspection findings and analysis. In most cases, corrosion indentations seem to be the cause of initiation of observed cracks, however, cyclical loading of the corroded members is the most probable cause for the propagation of the cracks. Further cyclical loading will likely lead to further crack propagation. Given the probable low toughness of existing steel, some cracks, if left unrepaired, might eventually reach the critical size that would cause the affected element to fracture. This risk is increased during cold weather as steel toughness is dependent upon temperature (Ref. section 2.6.6 of Calcul des charpentes d'acier, Picard et Beaulieu, ICCA/CISC 2003). There was no information about the steel toughness in our review of historical documentation. Performing Charpy impact tests on steel coupons from main members, stringers and floorbeams would provide useful information to evaluate of the criticality of observed cracking.

The following sub-sections present the significant findings of the inspection, along with an analysis of the defects found and associated recommendations, with associated BIM Priority Code.

4.5.1 FLOOBEAMS

No cracking was found in the distortion and constraint induced fatigue details of floorbeams. The only defects observed were the web out of plane deformations noted in earlier inspection reports. Floorbeams fatigue prone details (distortion and constraint induced) should be monitored at each fatigue inspection. Retrofitting the potential constraint induced fatigue details of the end plates by weld grinding would reduce the risk of cracking at those locations.

Recommendation:

Monitor location (end plates 3 weld intersection and web at stringer connection) at each fatigue inspection BIM Priority Code D

Retrofit the potential constraint induced fatigue details of the end plates by weld grinding BIM Priority Code C

4.5.2 STRINGERS

No cracks were detected on the stringer-to-floorbeam connecting angles. The angle radius and bolt holes periphery where inspected as described above.

A total of 39 cracks were found in the coped end of stringers. The exact location and length of cracking can be found in Appendix B.

Many of the locations tested randomly were found to be cracked, even there was no crack visible in the coating. In total, 39 cracks were identified, of which only 16 were found in previous inspections. Several adjacent stringers were found to be cracked, with crack lengths up to 25mm and, in some locations, cracks at both top and bottom copes (combined length of up to 45mm). Rotation of the stringers under live load can create a stress condition favorable to bottom cope crack growth, while restraint provided by the stringer-to-floorbeam bolted connection can create a condition favorable to top cope crack growth. Rotational rigidity of the bolted connection probably varies from one stringer to the other (bolt tension, bolt hole alignment, etc.), thus the level of rotation restraint is probably also variable. Given that most of the cracks were found on the bottom cope, it seems the condition favorable to bottom cope growth are more prevalent.

Figure 5: Typical Stringer Coped Ends Cracking – Stringer E, Floorbeam 14, Bay 14-16

Bays presenting consecutive stringers with cracks are as follows:

- Stringers in bay 0-2, at Floorbeam 0: 2 consecutive stringers with cracks lengths of 3mm to 15mm in bottom cope;
- Stringers in bay 2-4, at Floorbeam 2: 3 consecutive stringers with cracks lengths of 3mm to 10mm in bottom cope;
- Stringers in bay 6-8, at Floorbeam 6: 3 consecutive stringers with cracks lengths of 15mm in bottom cope;
- Stringers in bay 6-8, at Floorbeam 8: 2 consecutive stringers with cracks lengths of 5mm to 7mm in bottom cope;
- Stringers in bay 12-14, at Floorbeam 12: 3 consecutive stringers with cracks lengths of 3mm to 10mm in either top or bottom cope;
- Stringers in bay 14-16, at Floorbeam 14: 6 consecutive stringers with cracks lengths of 5mm to 20mm in bottom cope, 3 of which (consecutive) also have 10mm to 25mm of top cope; and
- Stringers in bay 14-16, at Floorbeam 16: 3 consecutive stringers with cracks lengths of 7mm to 15mm in bottom cope, 3 of which (consecutive) also have 3mm to 15mm of top cope.

Locations were several consecutive stringers are cracked might exist outside of those listed above, since not all stringers were tested.

Predicting the behavior of the cracked stringers is difficult given the unknown toughness of the steel, therefore, it is recommended to perform MT testing on all coped ends of steel stringers. It is suggested that this inspection be carried out prior to repairing existing cracks, so all cracks in a bay could be repaired in one mobilization to reduce costs. Bays with cracking in consecutive stringers should be repaired in the near future (BIM Priority Code A). Monthly MT inspections of bays with cracking in consecutive stringers is recommended to monitor the crack progression and assess/manage the risk level associated with the observed defect. Inspection access can be provided either by boat/barge, by rope access or by walking on the ice cover, if it forms under the bridge. In lieu of monthly MT testing, visual inspection from the top of the roadway should be undertaken to detect development of any potential major defects in stringers, such as the fracture of one web. The presence of sills on top of the stringers would be beneficial in the event of such fracture, since they would redistribute the load to the adjacent stringers. Plastic deformation in the sills is probable and could be detected by visual inspection from the deck. Early detection would help to prevent the subsequent overloading of adjacent stringers, the progression of existing cracks in adjacent stringers and would reduce the risk of potential fracture to multiple stringers.

Recommendation:

Perform systematic MT testing of all coped ends of stringers. BIM Priority Code A

Repair all cracks (drill crack tip and install tensioned ASTM A325 bolts or create radius as described in Chapter 5 of FHWA Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges) BIM Priority Code A

Monitor the location monthly before repairs and yearly after repairs, to assess the efficiency of repair BIM Priority Code A

4.5.3 MAIN TRUSS MEMBERS

Cracks of varying severity were found in several main members. An analysis of each defect found in the main members is presented in this section and is presented in the same order as presented in Appendix A.

- Gusset 16N: Linear perforation on inside gusset plate, with what might be cracking in the remaining corroding steel (see Figure 6). Defect parallel to shear stress from floorbeam. The member attached to gusset 16N (13N-16N) sees high stress range under bridge opening (112MPa at 63°) and moderate stress range under traffic loading (38MPa). Gusset plate with deterioration was analyzed by refined FEM method in 2015 (see “LaSalle Causeway Trunnion Joint Inspection and Analysis Report”; Report; MMM Group; June 5, 2015) and reinforcement was recommended. Observed deterioration (2018) has not progressed significantly since last gusset evaluation (2015). Deteriorated area is mainly in compression, but the criteria of CHBDC section 10.17.2.1 is not respected, hence fatigue cracking is possible in the gusset.

Recommendation:

Grind smooth the suspected crack and the perimeter of the perforation *BIM Priority Code A*

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element BIM Priority Code B

Figure 6: Linear Perforation in 16N Interior Plate, with Possible Crack

- **Main truss bottom chord 14S-15S:** Multiple cracks originating from perforation in batten plate near 15S. Crack lengths range from 3mm to 11mm in length. Crack are in secondary element of the member (batten plate) and cannot propagate to main elements of the members, because of the rivetted nature of the built-up section.

Recommendation:

Grind smooth the suspected crack and the perimeter of the perforation BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element BIM Priority Code C

Figure 7: Cracks Originating from Perforation in Batten Plate of 14S-15S

- **Main truss bottom chord 14S-16S:** 12mm long crack originating from perforation in web of inside Channel. Crack is perpendicular to main stress in member. Member composed of only 2 channels. Original 1916 steel, unknown toughness. Member undergoes stress reversal and tension under traffic load case, with moderate stress range (36 MPa under truck loading and it stays under compression for any current opening case). In our opinion, member 14S-16S is not a fracture critical member; it is a primary tension member, given the presence of member 14S-15S. Hence, the risk associated with the observed crack is reduced by the redundant load path.

Recommendation:

Grind smooth the crack and the perimeter of the perforation BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element BIM Priority Code A

Figure 8: 12mm Crack in 14S-16S Interior Channel's Web, Perpendicular to Main Stress and Near Perforation/Crack Like Indication, Parallel to Main Stress

- **Gusset 16S**: 3mm long crack, perpendicular to main stress on inside gusset plate, originating from a perforation in the plate. The member attached to gusset 16S (13S-16S) sees high stress range under bridge opening (126MPa at 63°) and moderate stress range under traffic loading (38MPa)

Recommendation:

Grind smooth the suspected crack and the perimeter of the perforation BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element BIM Priority Code C

Figure 9: 3mm Crack in 16S Interior Plate, Perpendicular to Main Stress

- **Main truss top chord 3S-5S**: 60mm long lamination with potential crack, mostly horizontal and parallel to main stress, in channel. 25mm area with multiple laminations or probable cracks, diagonal to main member stress. A metal half-moon shape disk, approx. 2-3mm thick, detached from the Channel. Observed defects located at the bottom corner of the Channel. Depth of defect unknown but most probably not superficial from visual observations. The top chord of the Bascule Bridge is under tension from dead load (the closed bridge is acting as a balanced cantilever with end barely touching). In 3S-5S, this tension is reduced during the opening cycle of the bridge (stress range of 15MPa at 63°) and even completely counterbalanced by vehicular loading (FLS #1 stress range of 41MPa). Based on discussions between Parsons Senior Bridge Engineers to assess the member sensitivity to fatigue from vehicular loading and the impact of the observed defect, it is our opinion that since a part of the stress range is in tension, fatigue cracking and crack propagation is possible under truck loading, even if the truck loading itself creates compression in 3S-5S. A crack can open each time the member is in tension after being compressed by the truck loading. The fact that the minimal value of stress in the stress range have almost no impact on the fatigue resistance is discussed in Reference Document “A Fatigue Primer for Structural Engineers”, Fisher, Kulak and Smith, NSBA, 1998.

As explained above, member 3S-5S undergoes stress reversal under each truck loading, with a moderate stress range of 41MPa. Even with a moderate stress range, an existing crack could propagate. The opinion of Senior NDT Technician Kent Leclair, of Brouco NDT about the left part of defect shown in Figure 10 is that there is “a strong possibility that the lamination has formed into a crack”. The probable low toughness of the steel channels of the top chord increases the possibility of a brittle fracture of the member in the event of crack propagation. Since the top chord is made two channels, a brittle fracture of one element could result in overloading the second channel beyond its capacity. The defect can be visually inspected from sidewalk with binoculars.

Recommendation:

Reinforce affected channel with slip critical bolted steel element BIM Priority Code A

Monitor defect monthly before repair and yearly after repair BIM Priority Code A

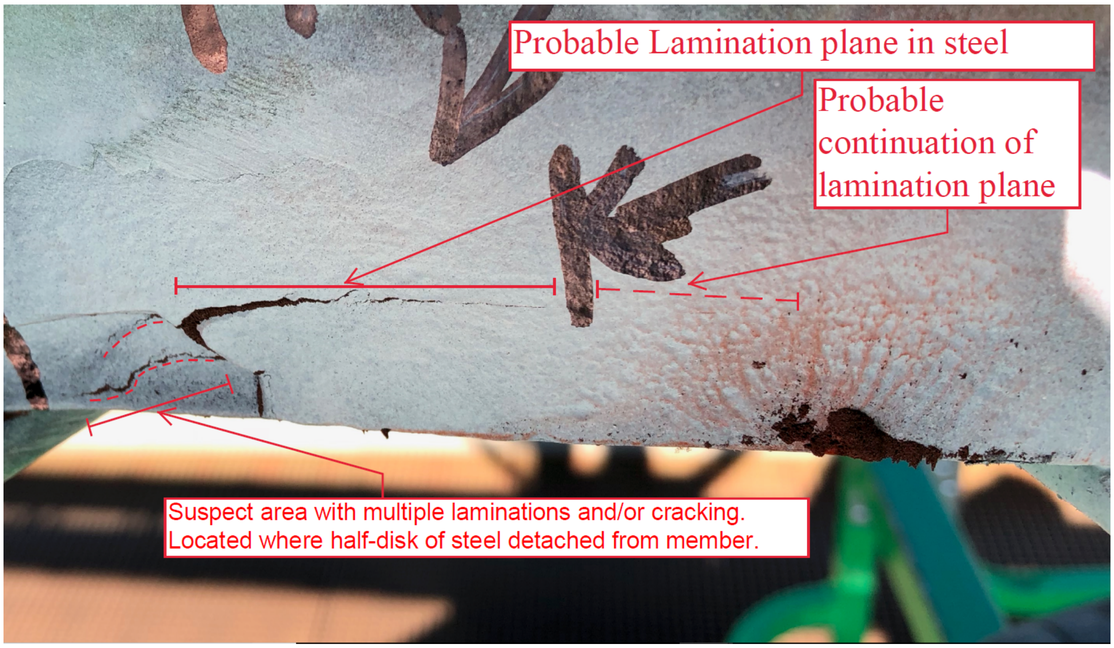


Figure 10: 60mm Lamination/Crack in 3S-5S Interior Channel

- **Main truss vertical 1N-2N:** 3mm crack, perpendicular to main stress, in flame cut hole. Member sees stress range of less than 6MPa under bridge opening to 63^{o} and of 25MPa under vehicular loading. Member is made of 5 elements (4 angles and one cut-out plate).

Recommendation:

Grind smooth the suspected crack or drill crack tip and install tensioned ASTM A325 bolt. BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code D

Figure 11: 3mm Crack Suspected, Perpendicular to Main Stress

- Main truss vertical 2N gusset: Full width horizontal hairline crack and multiple laminations in interior gusset. One lamination was found during the 2015 UT testing by Parsons. 2018 UT testing has revealed many laminations in the depth of the gusset plate and MT on the surface of the gusset has revealed a thin hairline crack on both visible (east and west) sides of the inside face. The location of the crack in the gussets is such that several bolts of the floorbeam end plate are located on each side of the cracked plane. Multiple laminations through the plate thickness makes retrofit by grinding impractical. Refined FEM analysis could indicate if strengthening of 2N interior gusset plate is necessary. Vertical member 1N-2N attached to gusset 2N sees stress range of less than 6MPa under bridge opening at 63°and of 25MPa under vehicular loading.

<u>*Recommendation:*</u>

Perform refined FEM analysis of the node to assess the need for strengthening. Assume inside gusset cracked at 100% in analysis. BIM Priority Code A

Repair gusset by plating, if required. BIM Priority Code A

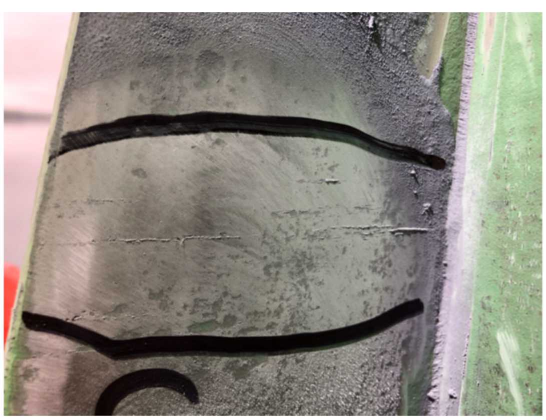


Figure 12 and Figure 13: Laminations and Hairline Crack, Perpendicular to Main Stress, in Interior Gusset Plate at 2N

- **Vertical 9S-10S:** 3mm crack, parallel to main stress, found in the heat affected zone (HAZ) of a removed weld (angle holding navigation sign).

Recommendation:

Remove angles holding sign to gain complete access to HAZ. Grind welds, HAZ and crack smooth. BIM Priority Code B

Perform Magnetic Particles testing of the HAZ zone, to check for cracking. Reinstate appropriate coating. BIM Priority Code B

Monitor the location yearly to assess the efficiency of crack grinding BIM Priority Code A

Figure 14 and Figure 15: 3mm Crack in HAZ Of Removed Weld on 9S-10S

- **Counterweight truss bottom chord 21N-27N:** Full width 100mm cracking of the horizontal flange of bottom interior angle. Crack is perpendicular to main stress. Appears to be through thickness. Crack was lost at inside radius when trying to find its end with MT testing. Angle is riveted to member's web plates, so direct crack propagation to web is impossible. Original 1916 steel, unknown toughness. Initiation might have been caused by corrosion. Member sees large stress ranges when bridge opens (157 MPa at 63°). Section loss due to cracking should be considered as the entire angle affected, because of high possibility of fracture of yet uncracked vertical leg of the angle. Strength calculations at ULS shows that the section 21N-27N at the location of the crack is overloaded under case B4 (CHDBC section 13.6.10.2). While the bridge is the closed position, member 21N-27N is under compression. Member is in tension when the bridge is open. Given the results of the demand to capacity calculations for the opening case (D/C of 1.09 for corresponding bending and tensile stresses under ULS B4 with fully cracked angle), yielding of other elements at the cracked section is possible under an extreme event. Furthermore, the increase in stress range caused by the loss of one angle could initiate new cracks or propagate existing cracks yet undetected. Hence, repair is recommended before the beginning of the new navigation season, in May 2019, to limit the number of opening with a damaged section in 21N-27N.

Recommendation:

Reinforce affected angle with slip critical bolted steel element BIM Priority Code A

Monitor defect monthly (if not repaired before May 2019) BIM Priority Code A

Figure 16 and Figure 17: Crack in 21N-27N Horizontal Flange of Bottom Interior Angle

- **Main truss diagonal 13N-16N**: 3mm crack found at impact damage, removed by grinding. Member susceptible to fatigue (see the Evaluation Section of this report). Crack was successfully removed by light grinding under the supervision of the Structural Engineer in charge of the fatigue inspection. MT testing was performed after grinding and no crack were found.

Recommendation:

Monitor the location annually to assess the efficiency of crack grinding. BIM Priority Code D

Figure 18 and Figure 19: 3mm Crack at Impact Damage (left), Removed by Light Grinding (right)

- **Tower truss horizontal member 15S-18S**: 3mm horizontal indication at the edge of channel's web perforation, possibly a crack initiation. Defect parallel to main stress. This member is composed of only 2 channels. Original 1916 steel, unknown resilience. Crack probably initiated by corrosion. Member sees limited tension stress range under bridge opening (6MPa at 63°) and vehicular traffic (3MPa).

Recommendation:

Grind smooth the suspected crack and the perimeter of the perforation. BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding. BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element. BIM Priority Code B

Figure 20: Perforation in 15S-18S Exterior Channel, with Possible 3mm Crack

- **Tower truss horizontal member 18S-19S**: 60mm vertical perforation with crack in the remaining material, perpendicular to main stress, at node 18S. This member is composed of only 2 channels. Original 1916 steel, unknown toughness. Crack probably initiated by corrosion. Member sees limited tension stress range under bridge opening (6MPa at 63º) and vehicular traffic (3MPa).

Recommendation:

Grind smooth the suspected crack and the perimeter of the perforation. BIM Priority Code A

Monitor the location yearly to assess the efficiency of crack grinding. BIM Priority Code D

Reinforce corroded element with slip critical bolted steel element. BIM Priority Code A

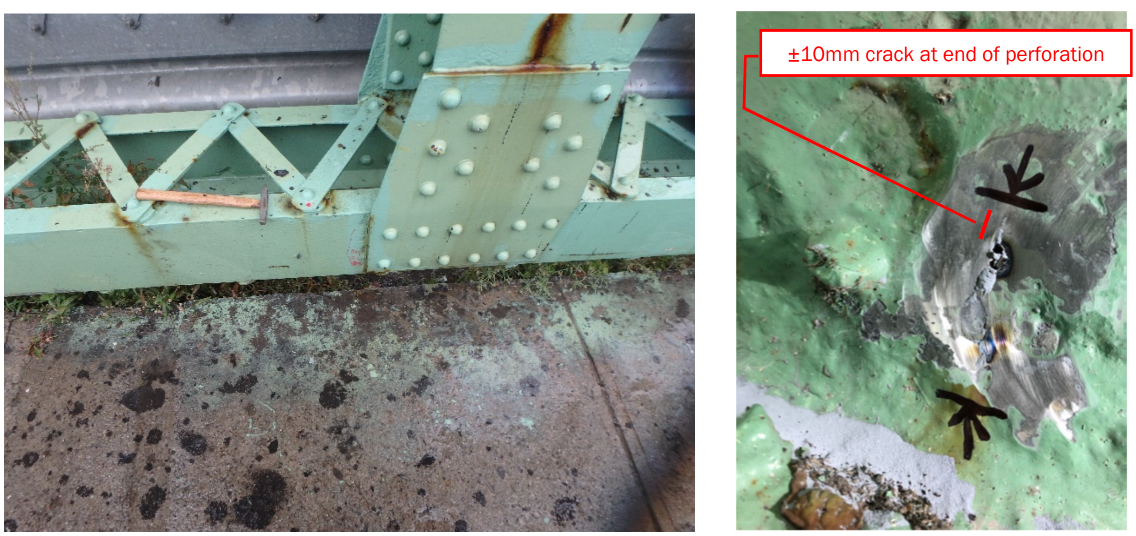


Figure 21: Linear Perforation in 18S-19S Exterior Channel, with Possible Crack

- Verticals 1-2, 5-6, 9-10, and 13-14, North and South

In 2016, cracks were discovered in the interior angles of vertical 13N-14N. Those cracks were repaired in 2017 by the addition of bolted steel elements. The cracks originated from a rivet hole, just above a discontinuity in the web plate of the vertical. Vertical 13S-14S was also reinforced in 2017, even if no cracks were detected in the member. In the current inspection, all similar details were inspected, and no cracks were detected. However, the detail where cracking occurred in 13N-14N is prone to concentrated stress in the angles and creates a high local stress range when the bridge is open or under vehicular traffic loading. Since the floorbeam connects to the interior gusset plate, the interior angles of the verticals most probably take a greater load than the outer angles. Secondary bending in the vertical caused by the connection of floorbeams to verticals, shear lag and bending stiffness change at discontinuity in web all contribute to concentrating stress in the angles at the location where cracking occurred in 13N-14N. Therefore, is would be beneficial to reinforce verticals presenting the same detail than 13-14 to prevent cracking.

<u>Recommendation:</u>

Reinforce the verticals 1-2, 5-6, 9-10 North and South to create continuity in web plate at lower end of member. BIM Priority Code C

Figure 22: Discontinuity in Verticals Webs

- Tack welds: Many tack welds or welded accessories (lamps, signs) were found on fracture critical or primary tension members. No cracks were found during the inspection, to the only exception of member 9S-10S, as described earlier.

<u>Recommendation:</u>

Grind welds smooth. Perform Magnetic Particles testing of the HAZ zone, to check for cracking. Reinstate appropriate coating. This activity can be performed at the same time than other repairs on the bridge, such as a general steel reinforcement or coating contract. BIM Priority Code C

4.5.4 COUNTERWEIGHT LINKS AND TRUNNIONS PINS

PAUT testing of the 8 main pins of the bridge was carried out according to the Inspection Plan. The Phased Array Specialist, (CGSB UT Level 2 Reg#13152, Phased Array UKAS Personnel Certification PCN#324214) inspected all 8 main pins following ASTM A388/A388M – 2010 Standard Practice for Ultrasonic Examination of Steel Forging. While many other historical bridges have cover plates to protect the end of their pin, the Bascule Bridge does not. This simplified the inspection, since no removal was needed.

Detailed results of the PAUT testing are included in Appendix D. No cracks were found during this inspection, but anomalies indicating potential surface corrosion were found on both counterweight trunnion pins at nodes 21S and 21N. Anomalies detected in UT inspection indicate that the surface is not uniform.

Recommendation:

Perform Phased Array Ultrasonic Testing (PAUT) to monitor progression of potential corrosion. BIM Priority Code C

Investigate corrosion mitigation options (penetrant rust inhibitor or other) BIM Priority Code B

4.5.5 FUTURE FATIGUE INSPECTIONS

Based upon the observed defects, the age of the structure and the volume of traffic on the bridge, it is suggested to perform biannual fatigue inspections. Such inspections could use a layered approach, with some members being inspected at each inspection and other every two or three inspections. Such layering could provide a cost-effective approach to fatigue inspections by targeting members and details according to their relative Fatigue Vulnerability. Targeting could be done following the method proposed in Section 5.7 Fatigue Vulnerability Assessment Tool.

All locations with known cracks, repaired or not, as well as all locations listed in the Inspection Tables should be monitored during the fatigue inspections.

To improve inspection and crack detection, the annual bridge cleaning should be coordinated with the inspection, to provide the cleanest surfaces possible to the inspectors. If this coordination is not possible, provision for targeted bridge cleaning immediately prior to fatigue inspection should be included in the contractual documents.

Recommendation:

Monitor locations of known cracking, repaired or not, at each fatigue inspection. BIM Priority Code D

Perform biannual fatigue inspection

Coordinate bridge cleaning and bridge inspection to have the best visual possible on all elements and ease crack finding. BIM Priority Code M

4.6 Summary of Fatigue Inspection Recommendations

Table 1 lists the recommendations of sections 4.5.1 to 4.5.4, as well as associated BIM Priority Code.

Table 1: Summary of Fatigue Inspection Recommendations

Member	Defect	Recommendation	BIM Priority Code	Traffic Impact
Stringers	Cracks at cope ends	*Perform systematic Magnetic Particles testing of all stringers coped ends*	A	None
		Repair all cracks (drill crack tip and install tensioned ASTM A325 bolts or create radius as described in Chapter 5 of FHWA Manual for Repair and Retrofit of Fatigue Cracks in Steel Bridges)	A	None
		Monitor the location monthly before repairs and yearly after repairs, to assess the efficiency of repair	A	None
Counterweight Trunnion Pins (21S and 21N)	*Surface anomalies indicating potential surface corrosion*	*Perform Phased Array Ultrasonic Testing (PAUT) to monitor progression of potential corrosion*	C	30 minutes closures
		Investigate corrosion mitigation options (penetrant rust inhibitor or other)	B	n/a
Bottom chord gusset 16N	*Possible cracking on inside gusset plate*	*Grind smooth the suspected crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	B	None
Main truss bottom chord 14S-15S	*Multiple cracks originating from perforation in cover/batten plate near 15S*	*Grind smooth the crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	C	None
Main truss bottom chord 14S-16S	*12mm crack originating from perforation in web of inside Channel*	*Grind smooth the crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	A	None
Bottom chord Gusset 16S	*3mm crack, perpendicular to main stress on inside gusset plate, originating from a perforation in the plate.*	*Grind smooth the suspected crack and the perimeter of the perforation.*	A	None
		Monitor the location yearly to assess the efficiency of crack grinding	D	None
		Reinforce corroded element with slip critical bolted steel element	C	None
Main truss top chord 3S-5S	*60mm lamination with potential crack, mostly horizontal and parallel to main stress, in channel. 25mm area with multiple laminations or probable cracks, diagonal to main member stress.*	*Monitor defect monthly before repair and yearly after repair*	*A*	*None or 30 minutes closures*
		Reinforce affected channel with slip critical bolted steel element	*A*	*1 Lane closure*
Main truss vertical 1N-2N	*3mm crack, perpendicular to main stress, in flame cut hole*	*Grind smooth the suspected crack or drill crack tip and install tensioned ASTM A325 bolt*	*A*	*1 Lane closure*
		Monitor the location yearly to assess the efficiency of crack grinding	*D*	*None or 30 minutes closures*
Main truss vertical 2N gusset	*Full width horizontal hairline crack and multiple laminations in interior gusset*	*Perform refined FEM analysis of the node to assess the need for strengthening. Assume inside gusset cracked at 100% in analysis*	*A*	*n/a*
		Repair gusset by plating, if required	*A*	*t.b.d*
Vertical 9S-10S	*3mm crack, parallel to main stress, found in the Heat Affected Zone (HAZ) of a removed weld (angle holding sign).*	*Remove angles holding sign to gain complete access to HAZ. Grind welds, HAZ and crack smooth.*	*B*	*30 minutes closures*
		Perform Magnetic Particles testing of the HAZ zone, to check for cracking. Reinstate appropriate coating.	*B*	*30 minutes closures*
		Monitor the location yearly to assess the efficiency of crack grinding	*A*	*None or 30 minutes closures*
Counterweight truss bottom chord 21N-27N	*Full width 100mm cracking of the horizontal flange of bottom interior angle. Crack is perpendicular to main stress. Appears to be through thickness.*	*Reinforce affected angle with slip critical bolted steel element*	*A*	*1 Lane closure*
		Monitor defect monthly (if not repaired before May 2019)	*A*	*None*
Main truss diagonal 13N-16N	*3mm crack found at impact damage, removed by grinding*	*Monitor the location yearly to assess the efficiency of crack grinding*	*D*	*30 minutes closures*

Table 1: Summary of Fatigue Inspection Recommendations (continued)

Member	Defect	Recommendation	BIM Priority Code	Traffic Impact
Tower truss horizontal member 15S-18S	*3mm horizontal indication at the edge of channel's web perforation, possibly a crack initiation*	*Grind smooth the suspected crack and the perimeter of the perforation.*	*A*	*None*
Tower truss horizontal member 15S-18S	*3mm horizontal indication at the edge of channel's web perforation, possibly a crack initiation*	*Monitor the location yearly to assess the efficiency of crack grinding*	*D*	*None*
Tower truss horizontal member 15S-18S	*3mm horizontal indication at the edge of channel's web perforation, possibly a crack initiation*	*Reinforce corroded element with slip critical bolted steel element*	*B*	*None*
Tower truss horizontal member 18S-19S	*60mm vertical perforation with crack in the remaining material, perpendicular to main stress, at node 18S*	*Grind smooth the suspected crack and the perimeter of the perforation.*	*A*	*None*
Tower truss horizontal member 18S-19S	*60mm vertical perforation with crack in the remaining material, perpendicular to main stress, at node 18S*	*Monitor the location yearly to assess the efficiency of crack grinding*	*D*	*None*
Tower truss horizontal member 18S-19S	*60mm vertical perforation with crack in the remaining material, perpendicular to main stress, at node 18S*	*Reinforce corroded element with slip critical bolted steel element*	*A*	*None*
Verticals 1-2, 5-6, 9-10, and 13-14, North and South	*Crack in 13N-14N, discovered in 2016, reinforced in 2017*	*Reinforce the verticals 1-2, 5-6, 9-10 North and South to create continuity in web plate at lower end of member*	*C*	*1 Lane closure*
All primary truss members	*Tack welds*	*Grind welds smooth. Perform Magnetic Particles testing of the HAZ zone,*	*C*	*1 Lane closure*
All primary truss members	*Tack welds*	*Reinstate appropriate coating*	*C*	*1 Lane closure*
All primary truss members	*Debris, spider webs, scale, etc.*	*Coordinate bridge cleaning and bridge inspection to have the best visual possible on all elements and ease crack finding.*	*M*	*30 minutes closures*
All floorbeams end plates	*Potential constraint induced fatigue details*	*Monitor location at each fatigue inspection*	*D*	*None*
All floorbeams end plates	*Potential constraint induced fatigue details*	*Retrofit the potential constraint induced fatigue details of the end plates by weld grinding*	*C*	*None*
All members	*Location with known cracks, repaired or not*	*Monitor location at each fatigue inspection*	*D*	*1 Lane closure*
Main members, stringers and floorbeams	*Unknown steel toughness*	*Perform Charpy testing on steel coupons*	*B*	*None*
Main members, stringers and floorbeams	*Fatigue Prone details*	*Perform Biannual fatigue inspections*	*n/a*	*1 Lane closure*

5.0 Fatigue Analysis

5.1 General

The fatigue evaluation was carried out based on the 3D CSI Bridge finite elements models and section properties (including net corroded section calculations) used in the 2017 Structural Evaluation. Stress ranges were calculated for both the traffic loading and the opening of the bridge in accordance with Sections 3, 13, and 14 of the CHBDC (see Figure 23). Fatigue limit state #1 was added to the 2017 models. Wind load cases acting on the opened bridge were also added to the models, with the intention of performing a sensitivity analysis of wind effects on fatigue life. Impact loading was applied to the stresses created by the opening of the bridge. A detailed methodology and results are presented in the following section of this report.

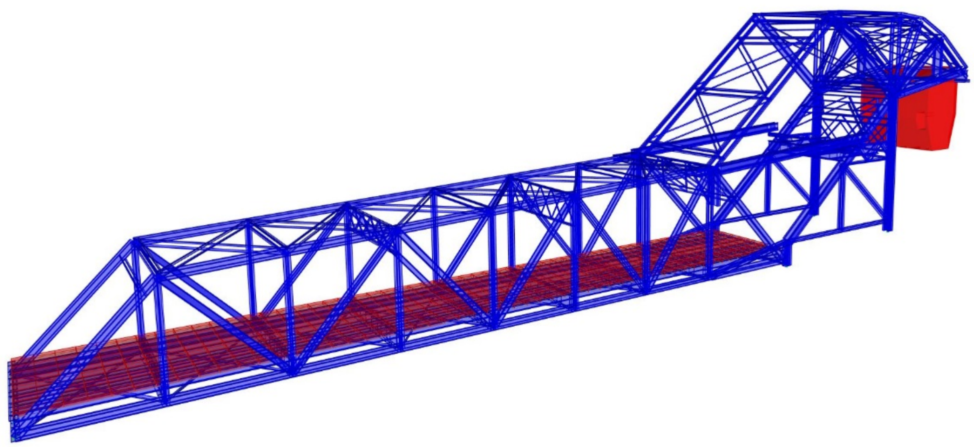


Figure 23: LaSalle Causeway Bascule Bridge Finite Element Model

The residual life of each primary truss member for both type of loading was estimated based on the methodology described in AASHTO Manual for Bridge Evaluation (3rd ed.) For the bridge case opening, the AASHTO formula was modified to account for the variation of number of openings per year in the 102-service life of the bridge. As required by Terms of Reference, results are presented separately for the traffic loading and the opening of the bridge and do not represent the total cumulative damage under all load cases. Such combination (Miner's law) is suggested as part of the development of the Fatigue Vulnerability Assessment Tool presented in Section 5.7.

The results of the fatigue evaluation show that the residual life values vary depending on the assumptions made for unknowns such as historical traffic data, angle of opening of the bridge, detail category of net section of riveted connections under fatigue loading, wind loading scenario for fatigue, etc. To address these assumptions, an upper and lower bound approach was chosen for the vehicular fatigue (closed position) evaluation and for the bridge opening fatigue evaluation. The comparison between the different residual life values allows the identification of the most vulnerable members and forms the basis for establishing priorities for intervention such as on-going fatigue inspection and preventive maintenance.

An example for an alternative approach to target fatigue vulnerabilities in the bridge is presented in the last section of the report to complement the traditional fatigue life calculation method required by the terms of reference.

5.1.1 RIVETED CONNECTION FATIGUE CATEGORY

For design purposes, net sections of riveted connections are considered as class D (Ref. CHBDC section 10.17), but, for fatigue evaluation analyses, they are considered class C if the member is in good condition (see AASHTO

MBE section 7.2.1 and C7.2.1). The explanation why a better fatigue performance is considered in evaluation is given in C7.2.1:

"For new design, the base metal at net sections of riveted connections is specified to be Category D. This represents the first cracking of a riveted member, which is highly redundant internally. Category C more accurately represents cracking that has propagated to a critical size. This increase in fatigue life for evaluation purposes is appropriate due to the redundancy of riveted members."

In summary, for evaluation purposes, the typical high redundancy of rivetted members allows for the crack initiation and propagation to a critical size. For the Bascule Bridge, many main truss members are effectively highly redundant internally. For instance, member 21-27 is composed of 8 elements, member 13-22 in composed of 6 elements, etc. However, some main members are made of only two elements, such as the top and bottom chords of the main trusses. With this low level of internal redundancy, accepting the crack growth to propagate to critical state is questionable.

Given the presence of members with high and low internal redundancy, the fatigue evaluation included the calculations for both Fatigue Detail Category C and D. A further analysis could use the internal redundancy of each member and match it with the most relevant Fatigue Detail Category. An example will be presented in Section 5.7 of this report.

5.1.2 SECTION LOSS AND STRUCTURAL REINFORCEMENTS

The 2017 structural evaluation accounted for section losses in the calculations of the residual bridge strength. Sections losses used in that report were based on previous inspections and on a site condition survey performed in 2017. The same section losses were considered in the 2018 fatigue evaluation. Areas where corrosion could have progressed since 2017 were closely inspected as part of the 2018 fatigue inspection and the data was updated for use in the fatigue evaluation. Given the age of the structure and location of observed cracks (section reduced by corrosion), net corroded sections were used in the Fatigue Evaluation.

5.2 Materials

In accordance with CHBDC Section 14, in lieu of original construction documents, strength of materials not showing signs of deterioration shall be determined based on test samples, date of construction or an approved method.

For this evaluation, since no material samples were extracted, therefore strength of materials was estimated based on date of construction according to clause 14.7.4 of CAN/CSA-S6-14 CHBDC.

5.2.1 STRUCTURAL STEEL

In the absence of other information, Table 14.1 of CHBDC states that for bridges constructed between 1905 and 1932, the specified yield strength of steel (F_y) shall be taken as 210 MPa and the ultimate strength (F_u) taken as 420 MPa. Since the Bascule Bridge was constructed between 1915 to 1917, the above values were taken as the strength of steel for the purposes of this evaluation.

Since all original main truss steel members except for those which have been reinforced or replaced, predate the toughness requirements specified in the applicable standards it is assumed that the toughness resistance is low. This has serious implications regarding crack propagation, especially in cold weather (Ref. section 2.6.6 of Calcul des charpentes d'acier, Picard et Beaulieu, ICCA/CISC 2003). No information about the steel toughness was found in our review of historical documentation. Performing Charpy impact tests on steel coupons from the main members, stringers and floorbeams would provide useful information to evaluate of the criticality of observed cracking.

Grade G40.12 steel is specified in the plans for stringers and floorbeams replaced in 1972 and 1973. The publication dates for G40.12 are 1964 and 1971, therefore, given the date of the replacement of the stringers, it is assumed that steel conforming to G40.12 1964 was used. For both the stringers and the floorbeams, toughness is unlikely to meet the current standards, as stated in the 2017 edition of Ministère des Transport du Québec Bridge Evaluation Manual, section 6.8.2, page 6-33:

"In Canadian standards, the first requirements regarding toughness appeared in 1974. [...] Steel made before 1978 thus rarely complies totally with current standards requirements regarding toughness".

Therefore, brittle behavior of a cracked stringer of floorbeam isn't to be excluded, especially in cold weather.

The unit weight of steel was taken as 77.0 kN/m^3 (unit mass of 7850 kg/m^3) in accordance with Table 3.4 of CHBDC.

5.2.2 CONCRETE

Plain concrete unit weight was taken as 23.5 kN/m^3 and reinforced concrete unit weight as 24.0 kN/m^3, in accordance with Table 3.4 of CHBDC.

5.3 Fatigue Evaluation for Vehicle Live Load

In the closed position, the bridge undergoes vehicular traffic loading as a typical fixed through-truss bridge. Under dead load only, the Bascule Bridge acts as a balanced cantilever. In order to have manageable mechanical stress when opening the bridge, the structure is balanced in such way that, in the closed position, the weight of the counterweight and its structure balances almost perfectly the weight of the through truss structure. Only a few kilonewtons are transferred to the abutment at node 0 under the dead load case. Hence, the 160ft span is acting as a cantilever for dead load. When additional load is added to the bridge, the structure behaves as a simple span truss and more load is transferred at abutment under node 0. The structural behavior under dead loads creates stresses of opposite sign to those created by the additional loads such as live loads. Therefore, the through-truss can be analyzed as a simple span truss, with an initial state of stress in all the main members. As the live load increases, the magnitude of stress in the members is reduced. If the live load is sufficiently increased, the state of stress in the members resulting from the dead load is completely cancelled by the live load and a stress reversal occurs.

CHBDC section 10.17.2.1 states the following: "At locations where the stresses resulting from the permanent loads are compressive, load-induced fatigue shall be disregarded when the compressive stress is at least twice the maximum tensile live load stress." This verification was performed for all members under live load and residual life was calculated only for those susceptible to structural fatigue under this provision. In the members where stresses resulting from the permanent loads are tensile and for which live loads create compressive stress, fatigue was assumed possible and the full stress range was used. In members where stress reversal occurs, some references suggest using only a portion of the compressive stress range. However, most North American references consulted, including CHDBC either do not include provisions to reduce the stress range in stress reversal members or indicate that the full stress range should be used. An in-depth discussion on this topic is presented at page 120 of the document "A Fatigue Primer for Structural Engineers" by John W. Fisher, Geoffrey L. Kulak, Ian F. C. Smith, National Steel Bridge Alliance, 1998

As per Terms of Reference, residual life calculations were performed according to AASHTO MBE 3rd Ed (2018) section 7.2. The formulae presented in this reference are the same as in CHBDC section 10.17, but with terms rearranged and some parameters added to allow for total fatigue detail life calculations in years. First, the fatigue detail is checked for Infinite Life (formula 7.2.4-1) and if the check fails the detail finite life is estimated with formula 7.2.5.1-1. This formula includes a provision to account for traffic variation during the lifespan of the bridge, by the use of logarithms and of a growth parameter (g). It also includes factor Rr to compute four levels of fatigue life. Section C7.2.5.1 gives an in-depth explanation about the differences in probability of failure for each level. The four levels are: Minimum (equivalent to CHBDC Section 10.17 Design life), Evaluation 1, Evaluation 2 and Mean. As stated in C7.2.5.1, Minimum or Evaluation 1 life is typically used to evaluate the fatigue serviceability of the detail.

7.2.5—Estimating Finite Fatigue Life

7.2.5.1—General

Four levels of finite fatigue life may be estimated:

- The minimum expected fatigue life (which equals the conservative design fatigue life),
- Evaluation 1 fatigue life (which equals a conservative fatigue life for evaluation),
- Evaluation 2 fatigue life (which equals a less conservative fatigue life for evaluation), and
- The mean fatigue life (which equals the statistically most likely fatigue life).

The total finite fatigue life of a fatigue-prone detail, in years, shall be determined as:

$$Y = \frac{\log\left[\frac{R_R A}{365n\left[(ADTT)_{SL}\right]_{PRESENT}\left[(\Delta f)_{eff}\right]^3} g(1+g)^{a-1}+1\right]}{\log(1+g)} \quad (7.2.5.1\text{-}1)$$

C7.2.5.1

Much scatter, or variability, exists in experimentally derived fatigue lives. For design, a conservative fatigue resistance of two standard deviations shifted below the mean fatigue resistance or life is assumed. This corresponds to the minimum expected finite fatigue life of this Article. Limiting actual usable fatigue life to this design fatigue life is most cautious and can be costly. As such, means of estimating the two evaluation fatigue lives and the mean finite fatigue life are also included to aid the evaluator in the decision making. Evaluation 1 is equivalent to the evaluation life in the previous specification, while Evaluation 2 fatigue life provides an additional choice for the user midway between Evaluation 1 and the mean fatigue life values.

Recent research has made it possible to obtain a closed-form solution for the total finite fatigue life using an estimated traffic growth rate and the present $(ADTT)_{SL}$. The estimated annual traffic growth rate can be obtained using available information in the agency's bridge inventory or the NBI. For cases with zero traffic growth, a very small value of g should be selected (less than 0.01 percent) for use in the expression for Y. Evaluators should be cautious in estimating traffic growth practically. For example an assumed *ADTT* in a single lane of 10,000 trucks per day corresponds to a truck passing every 8.6 seconds, around the clock 24/7. Clearly this would be an unrealistic assumption for many bridges.

Figure 24: Excerpt from AASHTO MBE 3rd Ed (2018)

Depending upon the combination of growth rate (g), Resistance Factor (R_R), stress range and daily truck traffic (ADTT SL Present), the residual life is either controlled by past growth rate or by future growth rate. In other words, for some combinations of R_R, stress range and ADTT SL Present, a growth rate equal to zero will lead to longer residual life, while for other combinations of R_R, stress range and ADTT SL Present the same growth rate of zero yields the exact opposite, with shorter residual life. This is explained in part by the long past service life of the bridge. Simulations for different values of g and ADTT SL Present were made to better understand the behavior of the formula under a growth rate variation. This was necessary to know what growth rates use for the calculations of a lower and upper bound of fatigue residual life.

Table 2: Effect of growth rate on Total Residual life (Y), for various ADTT SL Present

Total Detail Fatigue Life (Y, years)				
	ADDT SL Present (t=102 years)			
g	250	500	1000	5000
0.0005	223	114	57	11
0.005	210	131	75	16
0.05	151	136	122	87
0.07	142	131	121	96
0.08	139	129	120	98

The growth rate from 1990 to 2018 was calculated at g=0.83% from data obtained by PSPC from the City of Kingston. No data is available for the years 1916 to 1989. The MBE formula only allows for one growth rate to describe the whole service life of the bridge. If g= 0.83% is used, it means that from 1917 to the end of the bridge life, daily truck traffic has grown and will grow of 0.83% each year. Since the 0.83% growth rate was obtained from value for the last 28 years, it is reasonable to assume its validity to describe the future growth rate for the next few years. However, this cannot be said for the past growth rate. The 0.83% growth rate and the calculated Average Daily Truck Traffic in a Single Lane at the Present time (ADTT SL Present = 245, calculated from Kingston municipal data, 1990-2018) can be used to compute the ADTT SL of 1916 with the following formula: ADTT SL Present$*(1+g)^{-age}$ = $245*(1+0.0083)^{-102}$ = 106. That would mean a truck volume of 106*2 = 212 fatigue trucks per day on the bridge in 1916 Kingston which seems unrealistically high, given the population of Kingston in 1917 and the limited number of trucks on the roads in that period. Various possible past growth rates were used to try to obtain a plausible ADTT SL for 1916. A value of 2% was used and the associated ADTT SL for 1916 = $245*(1+0.02)^{-102}$ = 33. The results in Table 2 show that the 0.83% growth rate with an ADTT SL Present of 245 leads to the smallest conservative residual life. Thus, g=2% was used as the upper bound limit for residual life, while g=.83% was used for the lower bound. Better results could be obtained if the MBE formula could account for more than one growth rate. With more data for the past years, an accurate mean growth rate could be calculated and used in the actual formula. Given the short part of the bridge life for which data is available, the calculation of a mean growth rate could lead to a lot of uncertainties in the results, thus a lower and upper bound analysis was preferred.

5.3.1 LOAD CASES FOR EVALUATION FOR VEHICLE LIVE LOAD

Load cases for the fatigue evaluation under vehicle live load where taken from CHBDC, section 3. Fatigue limit state #1 load combination was added to the 2017 CSI Bridge Model.

- The following were included in the structural evaluation:
  - dead load (balanced bridge condition)
  - live load of vehicular traffic (fatigue)
- The following cyclical loadings were not included in the fatigue evaluation in the closed position:
  - sidewalk loading per CHBDC clause 14.9.5.1
  - wind load on traffic per CHBDC clause 14.9.5.3 (not deemed significant in the closed position)
  - wind load on structure per CHBDC clause 14.9.5.3 (not deemed significant in the closed position)

5.3.2 RESULTS OF EVALUATION FOR VEHICLE LIVE LOAD

Calculated residual life for all main members susceptible to load induced fatigue under vehicular live load are presented in Table 3 and Table 4. Results are presented from lower to higher residual fatigue life, for fatigue Detail Category C and D, as well as for Minimum (design), Evaluation 1, Evaluation 2 and Mean fatigue life levels. An upper bound of residual life was calculated with a growth rate of 2% and a lower bound with 0.83%.

The next pages present the residual life calculations results and figures showing the relative level of fatigue vulnerability of each member for a g=0.83%.

Table 3: Lower Bound of Member Residual Fatigue Life Under Vehicular Live Load

LOWER BOUND FOR MEMBER RESIDUAL LIFE UNDER TRAFFIC LOADING (Y-a, years)

ADTT_SL present	245	truck/day/lanes	Traffic growth (g)	0.0083		
Total span length	160	ft	Present brdige age (a)	102	years	
Number of lanes	2		Calculated ADTT SL 1916 ADDT SL present*$(1+g)^{-(a-1)}$	106	truck/day/lanes	
			Nb. Stress range per truck (n)	1		

Detail Cat.: C	Detail Cat.: D
Fsrt = 69MPa	Fsrt = 48 Mpa
Gamma = 1440*10^9	Gamma = 721*10^9

Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Det Cat C				Det Cat D			
			Fatigue Detail Category: C	Fatigue Detail Category: D	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)
4S-6S	561	fatigue possible	FINITE LIFE	FINITE LIFE	62	87	113	119	6	26	48	49
6S-8S	562	fatigue possible	FINITE LIFE	FINITE LIFE	66	90	117	123	9	29	51	52
4N-6N	281	fatigue possible	FINITE LIFE	FINITE LIFE	70	95	122	127	12	33	56	56
6N-8N	282	fatigue possible	FINITE LIFE	FINITE LIFE	73	98	125	131	15	35	58	59
9S-7S	489	fatigue possible	FINITE LIFE	FINITE LIFE	83	108	135	141	23	44	68	68
9N-7N	420	fatigue possible	FINITE LIFE	FINITE LIFE	84	110	137	143	24	45	69	70
7S-5S	490	fatigue possible	FINITE LIFE	FINITE LIFE	85	111	138	145	25	46	70	71
7N-5N	421	fatigue possible	FINITE LIFE	FINITE LIFE	87	113	141	147	27	48	72	73
10S-12S	564	fatigue possible	FINITE LIFE	FINITE LIFE	97	123	151	157	34	57	81	82
8S-9S	185	fatigue possible	FINITE LIFE	FINITE LIFE	99	125	153	159	36	58	83	84
8N-9N	9	fatigue possible	FINITE LIFE	FINITE LIFE	99	125	154	160	36	59	84	84
4S-1S	179	fatigue possible	FINITE LIFE	FINITE LIFE	99	126	154	160	36	59	84	85
4N-1N	3	fatigue possible	FINITE LIFE	FINITE LIFE	101	127	156	162	38	61	85	86
8S-10S	563	fatigue possible	FINITE LIFE	FINITE LIFE	103	129	157	163	39	62	87	88
10N-12N	284	fatigue possible	FINITE LIFE	FINITE LIFE	103	129	157	164	40	62	87	88
8N-10N	283	fatigue possible	FINITE LIFE	FINITE LIFE	111	138	166	172	46	70	95	96
3S-1S	492	fatigue possible	FINITE LIFE	FINITE LIFE	112	138	167	173	47	70	96	97
3N-1N	423	fatigue possible	FINITE LIFE	FINITE LIFE	114	140	169	175	49	72	98	98
5S-4S	183	fatigue possible	FINITE LIFE	FINITE LIFE	117	144	173	179	52	75	101	102
5S-3S	491	fatigue possible	FINITE LIFE	FINITE LIFE	118	145	174	180	52	76	102	103
5N-4N	7	fatigue possible	FINITE LIFE	FINITE LIFE	119	146	174	181	53	77	102	103
5N-3N	422	fatigue possible	FINITE LIFE	FINITE LIFE	120	147	176	182	54	78	104	105
5N-8N	8	fatigue possible	FINITE LIFE	FINITE LIFE	121	148	177	183	55	79	105	106
12S-13S	187	fatigue possible	FINITE LIFE	FINITE LIFE	137	165	194	200	69	94	120	121
16S-56S	399	fatigue possible	FINITE LIFE	FINITE LIFE	137	165	194	201	69	94	121	121
12N-13N	11	fatigue possible	FINITE LIFE	FINITE LIFE	138	166	195	201	70	94	121	122
16N-56N	248	fatigue possible	FINITE LIFE	FINITE LIFE	138	166	195	201	70	94	121	122
14S-16S	566	fatigue possible	FINITE LIFE	FINITE LIFE	155	184	213	220	85	111	138	139
14N-16N	286	fatigue possible	FINITE LIFE	FINITE LIFE	164	192	222	229	93	119	147	148
14S-13S	551	fatigue possible	INFINITE LIFE	FINITE LIFE					127	154	183	184
14N-13N	547	fatigue possible	INFINITE LIFE	FINITE LIFE					128	155	184	185
1S-0S	178	fatigue possible	INFINITE LIFE	FINITE LIFE					142	170	199	200
1N-0N	2	fatigue possible	INFINITE LIFE	FINITE LIFE					143	171	200	201
56S-13S	400	fatigue possible	INFINITE LIFE	FINITE LIFE					147	175	205	206
56N-13N	249	fatigue possible	INFINITE LIFE	FINITE LIFE					148	176	205	206
0N-2N	279	fatigue possible	INFINITE LIFE	FINITE LIFE					179	208	239	240

Table 3: Lower Bound of Member Residual Fatigue Life Under Vehicular Live Load (Continued)

LOWER BOUND FOR MEMBER RESIDUAL LIFE UNDER TRAFFIC LOADING (Y-a, years)

ADTT_SL present	245	truck/day/lanes	Traffic growth (g)	0.0083		Detail Cat.: C		Detail Cat.: D
Total span length	160	ft	Present brdige age (a)	102	years	Fsrt = 69MPa		Fsrt = 48 Mpa
Number of lanes	2		Calculated ADTT SL 1916 ADDT SL present*(1+g)^(a-1)	106	truck/day/lanes	Gamma = 1440*10^9		Gamma = 721*10^9
			Nb. Stress range per truck (n)	1				

Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Det Cat C				Det Cat D			
			Fatigue Detail Category: C	Fatigue Detail Category: D	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)
0S-2S	559	fatigue possible	INFINITE LIFE	FINITE LIFE					180	209	239	240
2S-4S	560	fatigue possible	INFINITE LIFE	FINITE LIFE					182	212	242	243
2N-4N	280	fatigue possible	INFINITE LIFE	FINITE LIFE					186	215	245	246
13S-11S	487	fatigue possible	INFINITE LIFE	FINITE LIFE					191	221	251	252
13N-11N	418	fatigue possible	INFINITE LIFE	FINITE LIFE					192	221	252	253
11S-9S	488	fatigue possible	INFINITE LIFE	FINITE LIFE					197	227	257	258
11N-9N	419	fatigue possible	INFINITE LIFE	FINITE LIFE					199	229	259	260
6S-5S	324	fatigue possible	INFINITE LIFE	FINITE LIFE					200	229	260	261
6N-5N	320	fatigue possible	INFINITE LIFE	FINITE LIFE					201	231	261	262
10S-9S	331	fatigue possible	INFINITE LIFE	FINITE LIFE					205	235	266	267
10N-9N	327	fatigue possible	INFINITE LIFE	FINITE LIFE					206	236	266	267
2S-1S	569	fatigue possible	INFINITE LIFE	FINITE LIFE					206	236	266	267
15N-16N	14	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18N-17N	19	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23N-24N	29	fatigue possible	INFINITE LIFE	INFINITE LIFE								
27N-25N	31	fatigue possible	INFINITE LIFE	INFINITE LIFE								
26N-25N	33	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23S-24S	157	fatigue possible	INFINITE LIFE	INFINITE LIFE								
24N-25N	160	fatigue possible	INFINITE LIFE	INFINITE LIFE								
26S-25S	164	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15S-16S	189	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18S-17S	194	fatigue possible	INFINITE LIFE	INFINITE LIFE								
24S-25S	206	fatigue possible	INFINITE LIFE	INFINITE LIFE								
27S-25S	207	fatigue possible	INFINITE LIFE	INFINITE LIFE								
2N-1N	241	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15N-18N	253	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18N-19N	254	fatigue possible	INFINITE LIFE	INFINITE LIFE								
22N-13N	431	fatigue possible	INFINITE LIFE	INFINITE LIFE								
22S-13S	435	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23N-22N	528	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23S-22S	530	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15S-18S	581	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18S-19S	582	fatigue possible	INFINITE LIFE	INFINITE LIFE								

Table 4: Upper Bound of Member Residual Fatigue Life Under Vehicular Live Load

UPPER BOUND FOR MEMBER RESIDUAL LIFE UNDER TRAFFIC LOADING
(Y-a, years)

ADTT_SL present	245	truck/day/lanes	Traffic growth (g)	0.02		Detail Cat.: C		Detail Cat.: D
Total span length	160	ft	Present brdige age (a)	102	years	Fsrt = 69MPa		Fsrt = 48 Mpa
Number of lanes	2		Calculated ADTT SL 1916 ADDT SL present*$(1+g)^{(a-1)}$	33	truck/day/lanes	Gamma = 1440*10^9		Gamma = 721*10^9
			Nb. Stress range per truck (n)	1				

Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Det Cat C				Det Cat D			
			Fatigue Detail Category: C	Fatigue Detail Category: D	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)
4S-6S	561	fatigue possible	FINITE LIFE	FINITE LIFE	57	70	83	86	24	36	49	50
6S-8S	562	fatigue possible	FINITE LIFE	FINITE LIFE	59	72	85	88	26	38	51	51
4N-6N	281	fatigue possible	FINITE LIFE	FINITE LIFE	61	74	87	90	28	41	53	54
6N-8N	282	fatigue possible	FINITE LIFE	FINITE LIFE	63	75	89	91	30	42	55	55
9S-7S	489	fatigue possible	FINITE LIFE	FINITE LIFE	68	81	94	97	34	47	60	60
9N-7N	420	fatigue possible	FINITE LIFE	FINITE LIFE	69	81	95	97	35	48	61	61
7S-5S	490	fatigue possible	FINITE LIFE	FINITE LIFE	69	82	95	98	36	48	61	62
7N-5N	421	fatigue possible	FINITE LIFE	FINITE LIFE	70	83	96	99	37	49	62	63
10S-12S	564	fatigue possible	FINITE LIFE	FINITE LIFE	75	88	101	104	41	54	67	67
8S-9S	185	fatigue possible	FINITE LIFE	FINITE LIFE	76	89	102	105	42	55	68	68
8N-9N	9	fatigue possible	FINITE LIFE	FINITE LIFE	76	89	102	105	43	55	68	69
4S-1S	179	fatigue possible	FINITE LIFE	FINITE LIFE	76	89	102	105	43	55	68	69
4N-1N	3	fatigue possible	FINITE LIFE	FINITE LIFE	77	90	103	106	44	56	69	70
8S-10S	563	fatigue possible	FINITE LIFE	FINITE LIFE	78	91	104	107	44	57	70	70
10N-12N	284	fatigue possible	FINITE LIFE	FINITE LIFE	78	91	104	107	44	57	70	70
8N-10N	283	fatigue possible	FINITE LIFE	FINITE LIFE	82	95	108	111	48	61	74	74
3S-1S	492	fatigue possible	FINITE LIFE	FINITE LIFE	82	95	109	111	49	61	74	75
3N-1N	423	fatigue possible	FINITE LIFE	FINITE LIFE	83	96	109	112	50	62	75	76
5S-4S	183	fatigue possible	FINITE LIFE	FINITE LIFE	85	98	111	114	51	64	77	78
5S-3S	491	fatigue possible	FINITE LIFE	FINITE LIFE	85	98	112	115	52	64	77	78
5N-4N	7	fatigue possible	FINITE LIFE	FINITE LIFE	86	99	112	115	52	65	78	78
5N-3N	422	fatigue possible	FINITE LIFE	FINITE LIFE	86	99	113	115	53	65	78	79
5N-8N	8	fatigue possible	FINITE LIFE	FINITE LIFE	87	100	113	116	53	66	79	79
12S-13S	187	fatigue possible	FINITE LIFE	FINITE LIFE	94	107	121	124	61	73	87	87
16S-56S	399	fatigue possible	FINITE LIFE	FINITE LIFE	95	108	121	124	61	73	87	87
12N-13N	11	fatigue possible	FINITE LIFE	FINITE LIFE	95	108	121	124	61	74	87	87
16N-56N	248	fatigue possible	FINITE LIFE	FINITE LIFE	95	108	121	124	61	74	87	87
14S-16S	566	fatigue possible	FINITE LIFE	FINITE LIFE	103	116	130	132	69	82	95	96
14N-16N	286	fatigue possible	FINITE LIFE	FINITE LIFE	107	120	134	136	73	86	99	100
14S-13S	551	fatigue possible	INFINITE LIFE	FINITE LIFE					90	103	116	116
14N-13N	547	fatigue possible	INFINITE LIFE	FINITE LIFE					90	103	116	117
1S-0S	178	fatigue possible	INFINITE LIFE	FINITE LIFE					97	110	123	124
1N-0N	2	fatigue possible	INFINITE LIFE	FINITE LIFE					97	110	124	124
56S-13S	400	fatigue possible	INFINITE LIFE	FINITE LIFE					99	112	126	126

Table 4: Upper Bound of Member Residual Fatigue Life Under Vehicular Live Load (Continued)

UPPER BOUND FOR MEMBER RESIDUAL LIFE UNDER TRAFFIC LOADING
(Y-a, years)

ADTT_SL present	245	truck/day/lanes
Total span length	160	ft
Number of lanes	2	

Traffic growth (g)	0.02	
Present brdige age (a)	102	years
Calculated ADTT SL 1916 ADDT SL present*(1+g)^-(a-1)	33	truck/day/lanes
Nb. Stress range per truck (n)	1	

Detail Cat.: C	Detail Cat.: D
Fsrt = 69MPa	Fsrt = 48 Mpa
Gamma = 1440*10^9	Gamma = 721*10^9

Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Det Cat C				Det Cat D			
			Fatigue Detail Category: C	Fatigue Detail Category: D	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)	Minimum residual life (Y-actual age)	Evaluation 1 residual life (Y-actual age)	Evaluation 2 residual life (Y-actual age)	Mean residual life (Y-actual age)
56N-13N	249	fatigue possible	INFINITE LIFE	FINITE LIFE					99	113	126	126
0N-2N	279	fatigue possible	INFINITE LIFE	FINITE LIFE					114	127	141	141
0S-2S	559	fatigue possible	INFINITE LIFE	FINITE LIFE					114	127	141	141
2S-4S	560	fatigue possible	INFINITE LIFE	FINITE LIFE					116	129	142	143
2N-4N	280	fatigue possible	INFINITE LIFE	FINITE LIFE					117	130	144	144
13S-11S	487	fatigue possible	INFINITE LIFE	FINITE LIFE					120	133	146	147
13N-11N	418	fatigue possible	INFINITE LIFE	FINITE LIFE					120	133	147	147
11S-9S	488	fatigue possible	INFINITE LIFE	FINITE LIFE					122	136	149	149
11N-9N	419	fatigue possible	INFINITE LIFE	FINITE LIFE					123	136	150	150
6S-5S	324	fatigue possible	INFINITE LIFE	FINITE LIFE					123	137	150	150
6N-5N	320	fatigue possible	INFINITE LIFE	FINITE LIFE					124	137	151	151
10S-9S	331	fatigue possible	INFINITE LIFE	FINITE LIFE					126	139	153	153
10N-9N	327	fatigue possible	INFINITE LIFE	FINITE LIFE					126	139	153	153
2S-1S	569	fatigue possible	INFINITE LIFE	FINITE LIFE					126	139	153	153
15N-16N	14	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18N-17N	19	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23N-24N	29	fatigue possible	INFINITE LIFE	INFINITE LIFE								
27N-25N	31	fatigue possible	INFINITE LIFE	INFINITE LIFE								
26N-25N	33	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23S-24S	157	fatigue possible	INFINITE LIFE	INFINITE LIFE								
24N-25N	160	fatigue possible	INFINITE LIFE	INFINITE LIFE								
26S-25S	164	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15S-16S	189	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18S-17S	194	fatigue possible	INFINITE LIFE	INFINITE LIFE								
24S-25S	206	fatigue possible	INFINITE LIFE	INFINITE LIFE								
27S-25S	207	fatigue possible	INFINITE LIFE	INFINITE LIFE								
2N-1N	241	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15N-18N	253	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18N-19N	254	fatigue possible	INFINITE LIFE	INFINITE LIFE								
17N-29N	269	fatigue possible	INFINITE LIFE	INFINITE LIFE								
22N-13N	431	fatigue possible	INFINITE LIFE	INFINITE LIFE								
22S-13S	435	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23N-22N	528	fatigue possible	INFINITE LIFE	INFINITE LIFE								
23S-22S	530	fatigue possible	INFINITE LIFE	INFINITE LIFE								
15S-18S	581	fatigue possible	INFINITE LIFE	INFINITE LIFE								
18S-19S	582	fatigue possible	INFINITE LIFE	INFINITE LIFE								

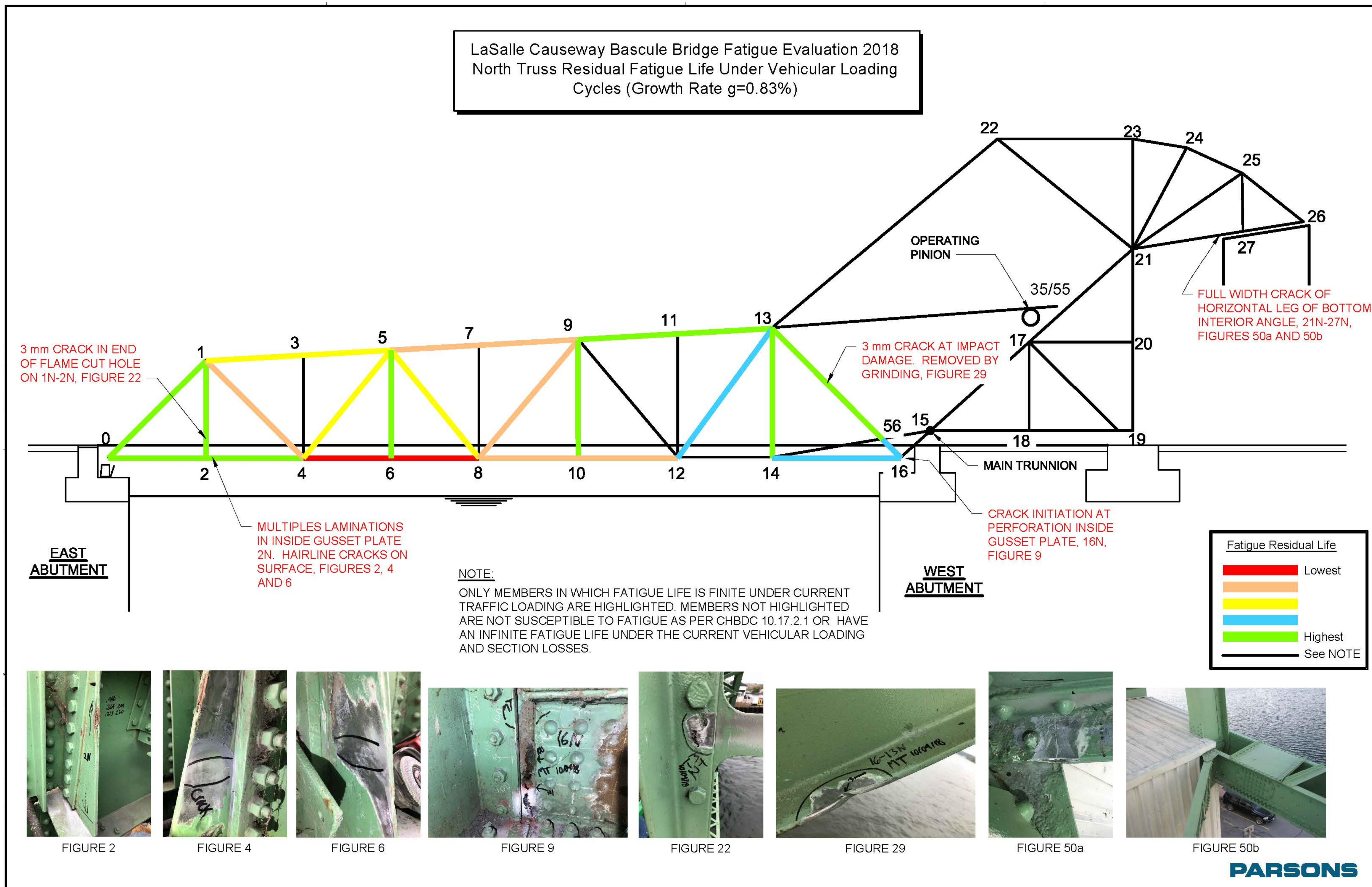


Figure 25: Fatigue Residual Life Under Vehicular Live Load – North Truss

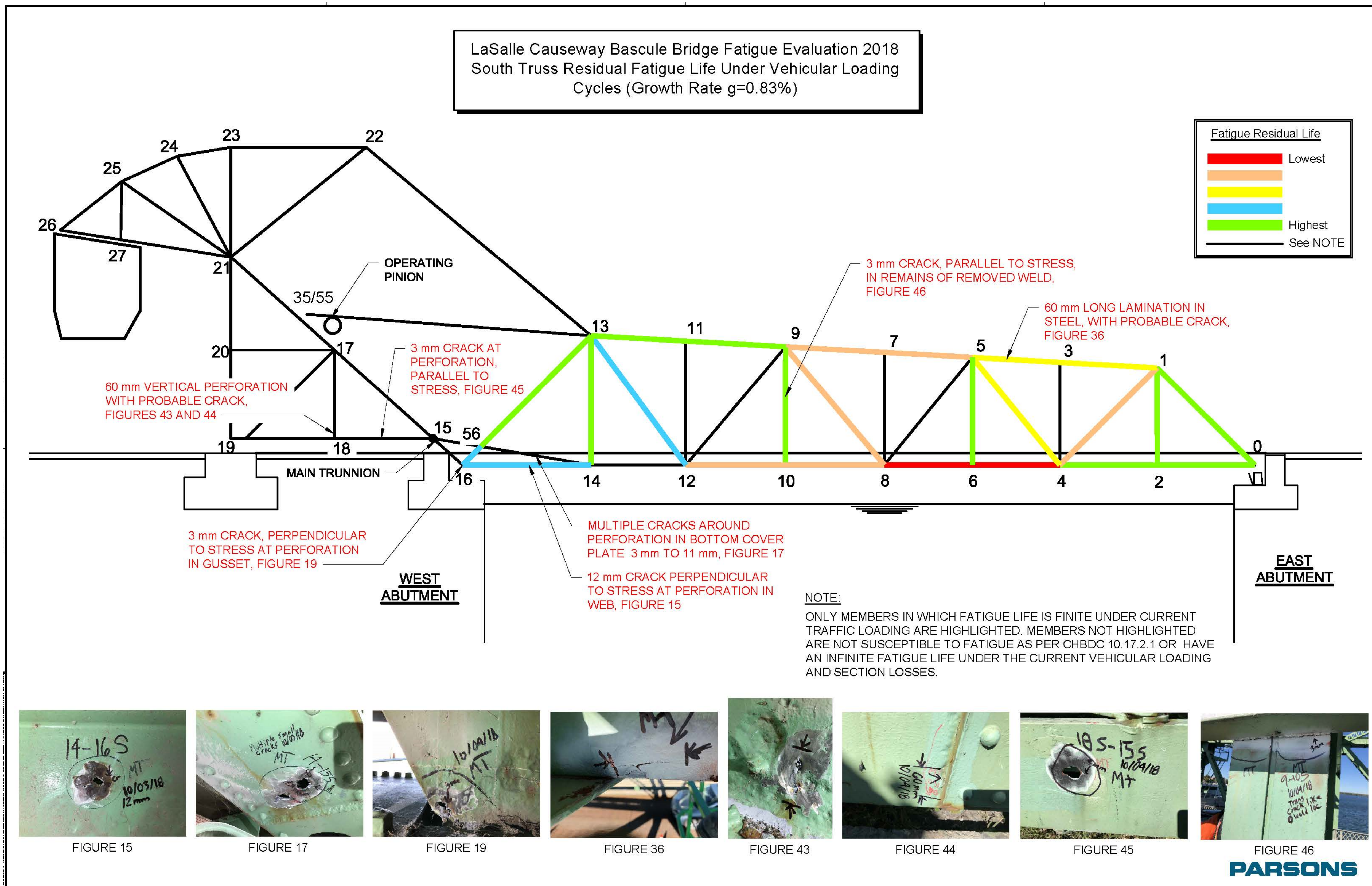


Figure 26: Fatigue Residual Life Under Vehicular Live Load – South Truss

5.3.3 ANALYSIS OF RESULTS OF EVALUATION FOR VEHICULAR LIVE LOAD

Fatigue evaluation of bridge primary truss members for vehicular live loads shows that few members are close to reaching the end of their theoretical fatigue life under the most conservative assumptions (g=.83%). All members have fatigue residual life of more than 24 years for less conservative but plausible assumptions (g=2%). In summary:

- No member has reached its theoretical Category C or D fatigue life for any level of fatigue life (Minimum, Evaluation 1, Evaluation 2 or Mean) and for any of the traffic growth rate scenario considered (g=0.83% or g=2%)
- Four members are within 15 years of reaching the theoretical end of their Category D fatigue life for the conservative Minimum level (equivalent to design life), when the 0.83% traffic growth rate scenario in considered. This scenario gives the lower bound of fatigue life. These members are:
  - 4S-6S, 6S-8S, 4N-6N and 6N-8N
- All members have a Category C or D fatigue residual life of more than 24 years for any of the traffic growth rate scenario considered (g=0.83% or g=2%) for fatigue live level Evaluation 1, Evaluation 2 or Mean.

This analysis allows to target members that are theoretically the most vulnerable to cracking in the next years. Regular targeted fatigue inspections allow for early crack detection in these members. Preventive maintenance to remove sharp indentations created by corrosion reduces the risk of cracking in the most vulnerable members. Preventing further section loss in most vulnerable members is also recommended. If more section loss occurs, the stress range will increase, and residual life will be negatively affected. Recommendations pertaining to this evaluation are summarized in Section 5.6.

5.4 Fatigue Evaluation for Bridge Opening Cycle

The opening cycle of the Bascule Bridge causes significant changes to the stresses in all members and therefore, a large stress range for many structural members under evaluation. As described in Section 5.3, the 160ft span acts as a cantilever under dead load. As the bridge opens, the bending moment in the main truss structure decreases as the bridge gets closer to the vertical position resulting in compressive stresses in the main truss as it acts more like a column than a cantilevered girder. In the counterweight truss, the effect of the opening cycle is also influenced by the position of the counterweight. As the bridge opens, the relative position of the counterweight center of gravity changes compared to the counterweight truss. The bending moment that the counterweight truss must resist this change in orientation and this creates a stress reversal in most member directly attached to the counterweight. For example, counterweight bottom chord member 21-27 is in compression when the bridge is closed and in tension when it is open. Figures 28 - 30 show the bridge in the open 84^{o}, open 63^{o} and closed positions, respectively the current maximum opening angle and the original opening angle (see Section 5.4.1 for more detail). Those three positions were used to calculate the stress ranges from the bridge opening cycle.

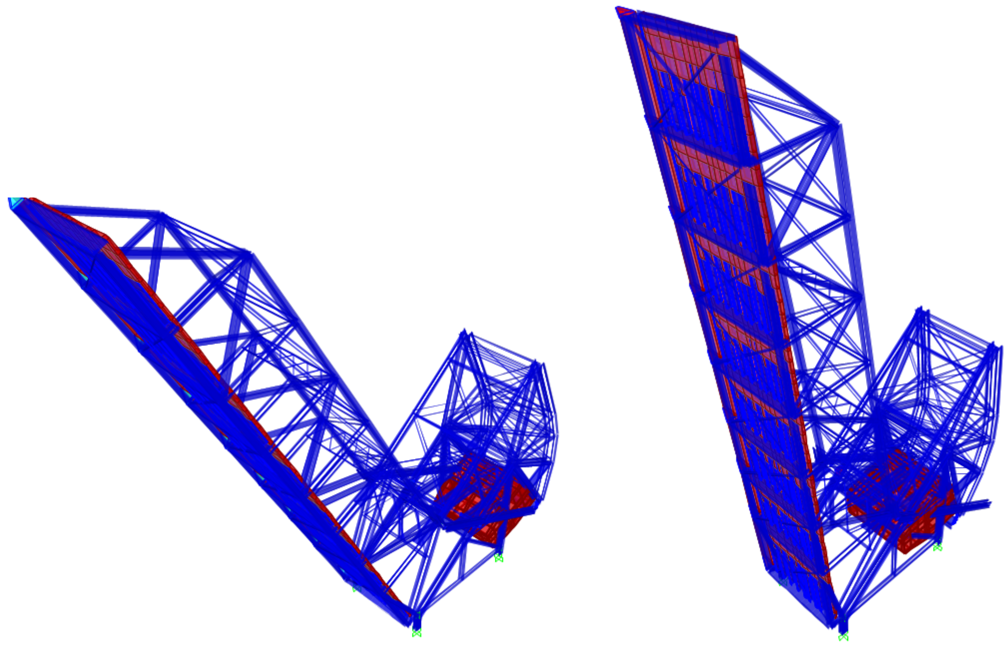


Figure 27: Screenshot of LaSalle Causeway Bascule Bridge CSI Bridge Finite Element Model at 63o and 84o opening, from 2017 Structural Evaluation

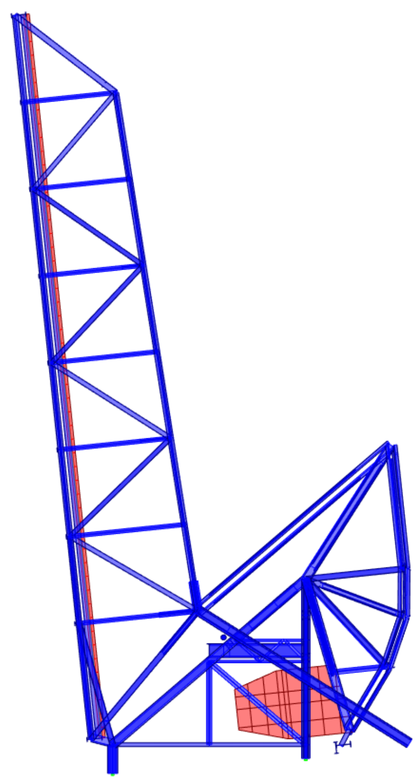


Figure 28: Screenshot of LaSalle Causeway Bascule Bridge Finite Element Model- 84o Opening – North View

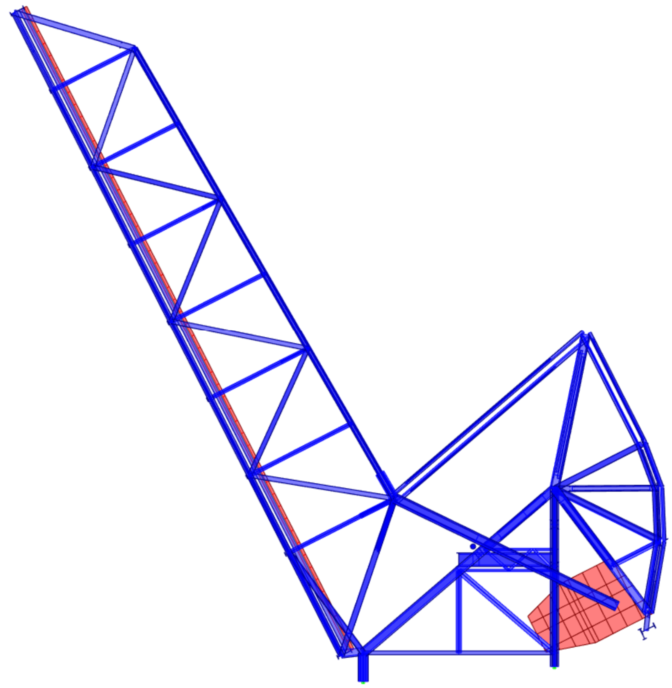


Figure 29: Screenshot of LaSalle Causeway Bascule Bridge Finite Element Model- 63o Opening – North View

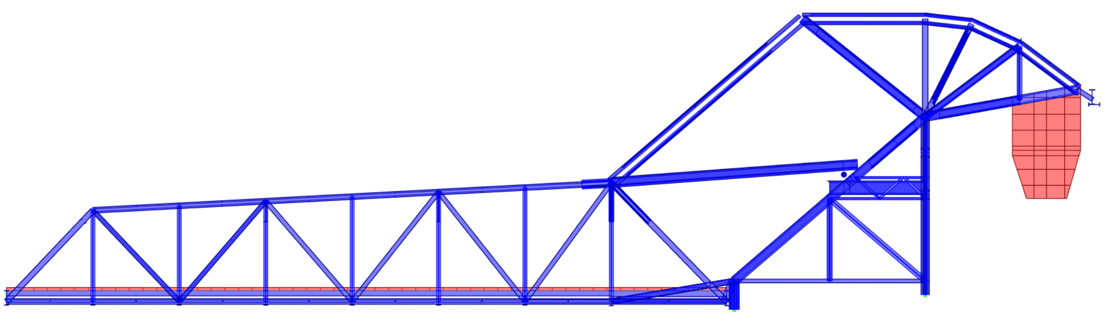


Figure 30: Screenshot of LaSalle Causeway Bascule Bridge Finite Element Model- Closed Position – North View

5.4.1 OPENING ANGLE FOR STRESS RANGE CALCULATIONS

As discussed in Section 5.4, the bridge opening angle has a major impact on the stress range and hence on the residual fatigue life. From its original design, the maximum operation angle of the bridge is 84°. Historically, the bridge was operated manually and was not opened more than needed to allow the passage of boat traffic, hence it is unlikely that every opening reached 84°

At some point, modifications were made which limited the maximum angle the bridge can reach. It is now of 65°. as confirmed in an email from PSPC bridge operations to Parsons dated June 27, 2017. An interference between the counterweight and the guardrails seems to be the limiting element. A few years ago, the bridge opening was automated, and it now opens to 65o each time.

Since the bridge angle has a major influence on the stress range in members and that there is no known record of past bridge opening angle, there is a large uncertainty on the stress ranges really experienced by the bridge until the opening was automated to systematically reach 65°. The possibility of many opening cycles reaching 84° prior to automation cannot be ruled out but considering that all past openings have reached such a large angle would be overly conservative. Moreover, 84° will not be reached for any future openings because of automation and physical interference.

This situation led to the choice of an upper and lower bound analysis for bridge opening angle. Bridge opening angles of 63° [2] and 84° degrees were used to calculate residual fatigue life and results are presented in the following sections of this report.

5.4.2 NUMBER OF OPENING CYCLES

The actual number of past opening was based on records from PSPC and from the 1997 Stringer Report, which presented an estimate of the number of opening cycles up to 1997. This estimate was used in the subsequent reports and is regarded as reliable. The number of openings per year before 1997 varied significantly from year to year, with up to 2890 openings per year. The number of openings since 1997 was estimated from PSPC records. It is estimated that the bridge has opened approximately 193 000 times since its construction and that it will open 900 times per year for the upcoming years.

5.4.3 OPERATING IMPACT LOAD

According to CHBDC clause 13.6.10.2, an operating impact load (I_o) of 20% was applied to the maximum dead load effect in all members that are in motion and to the load effect on a stationary member caused by the moving dead load. Due to the nature of impact forces, this load was applied in either upward or downward directions to determine the maximum effect.

It is acknowledged that the code prescribed factors may be overly conservative and more applicable to setting the standard for new bascule bridge construction rather than evaluating a structure that has already operated for 100 years. The 1997 D. Stringer Fatigue Evaluation report states that the actual impact load was closer to 9%. This value was based on strain gauge measurements during the bridge opening. It is known that some extreme events happened in the life of the bridge, events in which the impact load was most probably more than 9%, but for a fatigue load case, the most prevalent condition should be used to get realistic results. Hence, an impact factor of 10% was used in the residual life calculations for the upper bound scenario. The code 20% impact factor was used for the lower bound scenario.

5.4.4 SPECIAL LOAD CASES FOR BASCULE BRIDGES

Special load cases for bascule bridges per Section 13 of the CHBDC were not applied to the bridge structure for the fatigue evaluation, since those loads are assumed not to be occurring at each opening of the bridge, except for the impact load described in the previous section. Wind loading pressures corresponding to return periods relevant for sensitivity evaluation of wind effect on fatigue life were also considered. The choice of return period used is described in the following sections of this report.

5.4.5 WIND LOADING

Wind can have an impact on the fatigue life of a bridge, depending on its characteristics. The 2017 structural evaluation has shown the vulnerability of the bridge in the open position to design wind pressures. Therefore,

[2] The bridge was modeled to 63° in the 2017 Structural Evaluation, as bridge was evaluated under 5 different positions 0°, 21°, 42°, 63 ° and 84 °. Stress range variation between 63 ° degree and 65 ° is marginal, so values from 2017 Structural Evaluation for an opening of 63 ° were used in the current report calculations.

the effects of wind loads on fatigue life were studied in the fatigue evaluation to determine if they have a significant effect on fatigue life. In lieu of any guidelines for consideration of wind effects for fatigue evaluations of an opened bascule bridge the assumption that a wind with a recurrence period close to the frequency of bridge opening cycle was considered appropriate. Hence, CHBDC design wind pressures are not considered suitable since they represent longer return periods. In lieu of a wind study to determine the appropriate wind pressures to use for the fatigue evaluation, wind pressures of 350Pa (85 km/h wind speed) and 520Pa (150 km/h wind speed) were assumed which represent the unfactored wind pressure in Kingston for 10 and 100 year return period respectively. For comparison, the design wind pressure for a bascule bridge would be 1500Pa (CHBDC section 13.6.4.9). The calculations were completed for the 63º and 84º case, with and without wind. In all cases, the effect of wind on the residual fatigue life was found to be marginal. For instance, in the 84º opening case, the worst wind effect creates a stress range of approximately 15MPa in member 11N-13N, while the opening itself creates a stress range of approximately 182MPa. Using a lower and more representative value of fatigue wind pressure would yield even lower stress ranges associated with the wind fatigue load case.

Since the wind load effects were found to have a marginal impact on the fatigue stress range for the open cycle, the wind load cases were excluded for the calculation presented in Table 5 and Table 6.

5.4.6 RESULTS OF EVALUATION FOR BRIDGE OPENING CYCLES

Calculated residual life for all main members susceptible to load induced fatigue under bridge opening are presented in Table 5 and Table 6. Results are presented from lower to higher residual fatigue life, for Fatigue Detail Category C and D. The Upper bound scenario for residual fatigue life was calculated for a 63º opening angle, 10% impact load and no wind. The lower bound scenario for residual fatigue life was calculated for an 84º opening angle, 20% impact load and no wind loading cases.

For almost all main truss members, the highest stress ranges are created when the bridge is open to its maximum. This implies there is only one stress cycle for each bridge opening and closing, as noted in 1998 Fatigue Probabilistic Assessment of the LaSalle Causeway Bascule Bridge. However, members 13N-56N and 13S-56S are different, since the stress reaches a peak with the bridge at a 45º open angle position. With a maximum opening to 84º, the member undergoes two stress cycles for each opening. However, since the opening is currently limited to 65º, the difference between the peak force generated at 45º opening and the force at 65º opening is small. Analysis showed that those members have residual life in excess of 100 years for the current 65º opening, even though the 45º opening peak stress effect is accounted for.

The next pages present the residual life calculations results and figures showing the relative level of fatigue vulnerability of each member for the Upper bound scenario.

Table 5: Upper Bound of Member Residual Fatigue Life Under Bridge Opening Cycle

MEMBER RESIDUAL LIFE UNDER BRIDGE OPENING CYCLE TO 63° (years)						
Upper bound : all variables to minimum values						
Number of opening cycles since construction		193120	Impact factor: 10%		No wind	
Number of opening per year, future years (Nc_yr)		900	Opening: 63°		Net section and corrosion taken into account	
Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check ($\Delta\sigma$ opening < Fsrt/2)		Finite Life check (years remaining at Nc_yr openings per year)	
			Fatigue Detail Category: C	Fatigue Detail Category: D	Fatigue Detail Category: C	Fatigue Detail Category: D
26N-25N	33	Fatigue possible	FINITE LIFE	FINITE LIFE	177	-9
26S-25S	164	Fatigue possible	FINITE LIFE	FINITE LIFE	183	-8
27N-21N	277	Fatigue possible	FINITE LIFE	FINITE LIFE	197	-4
27S-21S	608	Fatigue possible	FINITE LIFE	FINITE LIFE	200	-3
16S-56S	399	Fatigue possible	FINITE LIFE	FINITE LIFE	587	187
26N-27N	276	Fatigue possible	FINITE LIFE	FINITE LIFE	826	306
26S-27S	607	Fatigue possible	FINITE LIFE	FINITE LIFE	833	310
16N-56N	248	Fatigue possible	FINITE LIFE	FINITE LIFE	921	354
24N-25N	160	Fatigue possible	FINITE LIFE	FINITE LIFE	987	387
24S-25S	206	Fatigue possible	FINITE LIFE	FINITE LIFE	999	393
23N-24N	29	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	472
23S-24S	157	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	480
56S-13S	400	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	723
13N-11N	418	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	748
11N-9N	419	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	765
13S-11S	487	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	845
23N-22N	529	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	850
23S-22S	531	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	865
11S-9S	488	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	879
5N-4N	7	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
8N-9N	9	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
12N-13N	11	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
27N-25N	31	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
8S-9S	185	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
12S-13S	187	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
27S-25S	207	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
56N-13N	249	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
9N-7N	420	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
7N-5N	421	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
22N-13N	431	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
22S-13S	435	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
9S-7S	489	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
7S-5S	490	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000

Table 5: Upper Bound of Member Residual Fatigue Life Under Bridge Opening Cycle (Continued)

MEMBER RESIDUAL LIFE UNDER BRIDGE OPENING CYCLE TO 63° (years)						
Upper bound : all variables to minimum values						
Number of opening cycles since construction		193120	Impact factor: 10%		No wind	
Number of opening per year, future years (Nc_yr)		900	Opening: 63°		Net section and corrosion taken into account	
Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Finite Life check (years remaining at Nc_yr openings per year)	
			Fatigue Detail Category: C	Fatigue Detail Category: D	Fatigue Detail Category: C	Fatigue Detail Category: D
14N-13N	547	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
14S-13S	551	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
5S-4S	183	Fatigue possible	INFINITE LIFE	FINITE LIFE		>1000
1N-0N	2	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18N-17N	19	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
13N-35N	27	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
13S-35S	142	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
1S-0S	178	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18S-17S	194	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
2N-1N	241	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15N-18N	253	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18N-19N	254	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
6N-5N	320	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
6S-5S	324	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
10N-9N	327	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
10S-9S	331	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
5N-3N	422	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
3N-1N	423	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
5S-3S	491	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
3S-1S	492	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
2S-1S	569	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15S-18S	581	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18S-19S	582	Fatigue possible	INFINITE LIFE	INFINITE LIFE		

Table 6: Lower Bound of Member Residual Fatigue Life Under Bridge Opening Cycle

MEMBER RESIDUAL LIFE UNDER BRIDGE OPENING CYCLE TO 84° (years)						
Lower bound : all variables to maximum values						
Number of opening cycles since construction		193120	Impact factor: 20%		No wind	
Number of opening per year, future years (Nc_yr)		900	Opening: 84°		Net section and corrosion taken into account	
Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Finite Life check (years remaining at Nc_yr openings per year)	
			Fatigue Detail Category: C	Fatigue Detail Category: D	Fatigue Detail Category: C	Fatigue Detail Category: D
26S-25	164	Fatigue possible	FINITE LIFE	FINITE LIFE	-37	-70
26N-25N	33	Fatigue possible	FINITE LIFE	FINITE LIFE	-37	-70
27S-21S	608	Fatigue possible	FINITE LIFE	FINITE LIFE	-17	-59
27N-21N	277	Fatigue possible	FINITE LIFE	FINITE LIFE	-16	-59
24S-25S	206	Fatigue possible	FINITE LIFE	FINITE LIFE	46	-40
24N-25N	160	Fatigue possible	FINITE LIFE	FINITE LIFE	47	-40
23S-24S	157	Fatigue possible	FINITE LIFE	FINITE LIFE	79	-32
23N-24N	29	Fatigue possible	FINITE LIFE	FINITE LIFE	82	-31
13N-11N	418	Fatigue possible	FINITE LIFE	FINITE LIFE	167	-11
11N-9N	419	Fatigue possible	FINITE LIFE	FINITE LIFE	177	-9
11S-9S	488	Fatigue possible	FINITE LIFE	FINITE LIFE	180	-8
13S-11S	487	Fatigue possible	FINITE LIFE	FINITE LIFE	184	-7
26S-27S	607	Fatigue possible	FINITE LIFE	FINITE LIFE	192	-5
26N-27N	276	Fatigue possible	FINITE LIFE	FINITE LIFE	194	-5
23S-22S	530	Fatigue possible	FINITE LIFE	FINITE LIFE	246	16
23N-22N	528	Fatigue possible	FINITE LIFE	FINITE LIFE	252	19
21N-17N	262	Fatigue possible	FINITE LIFE	FINITE LIFE	463	125
22N-13N	431	Fatigue possible	FINITE LIFE	FINITE LIFE	472	129
22S-13S	435	Fatigue possible	FINITE LIFE	FINITE LIFE	474	130
12S-13S	187	Fatigue possible	FINITE LIFE	FINITE LIFE	482	134
21S-17S	590	Fatigue possible	FINITE LIFE	FINITE LIFE	496	141
22N-21N	424	Fatigue possible	FINITE LIFE	FINITE LIFE	626	206
22S-21S	601	Fatigue possible	FINITE LIFE	FINITE LIFE	643	215
12N-13N	11	Fatigue possible	FINITE LIFE	FINITE LIFE	707	247
8S-9S	185	Fatigue possible	FINITE LIFE	FINITE LIFE	739	263
8N-9N	9	Fatigue possible	FINITE LIFE	FINITE LIFE	982	384
9N-7N	420	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	451
7N-5N	421	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	510
9S-7S	489	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	605
9S-12S	186	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	662
7S-5S	490	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	664
5S-8S	184	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	773
9N-12N	10	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	918
23S-21S	583	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	948
23N-21N	255	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	952
5N-8N	8	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	956

Table 6: Lower Bound of Member Residual Fatigue Life Under Bridge Opening Cycle (Continued)

MEMBER RESIDUAL LIFE UNDER BRIDGE OPENING CYCLE TO 84° (years)						
Lower bound : all variables to maximum values						
Number of opening cycles since construction		193120	Impact factor: 20%		No wind	
Number of opening per year, future years (Nc_yr)		900	Opening: 84°		Net section and corrosion taken into account	
Member	CSI #	CHBDC 10.17.2.1 Fatigue check required?	Infinite Life check (Δσ opening < Fsrt/2)		Finite Life check (years remaining at Nc_yr openings per year)	
			Fatigue Detail Category: C	Fatigue Detail Category: D	Fatigue Detail Category: C	Fatigue Detail Category: D
14N-16N	286	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
14S-16S	566	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
16N-56N	248	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
16S-56S	399	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
56S-13S	400	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
5N-4N	7	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
27N-25N	31	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
27S-25S	207	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
56N-13N	249	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
14N-13N	547	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
14S-13S	551	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
5S-4S	183	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
5N-3N	422	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
4N-1N	3	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
4S-1S	179	Fatigue possible	FINITE LIFE	FINITE LIFE	>1000	>1000
3N-1N	423	Fatigue possible	INFINITE LIFE	FINITE LIFE		>1000
5S-3S	491	Fatigue possible	INFINITE LIFE	FINITE LIFE		>1000
3S-1S	492	Fatigue possible	INFINITE LIFE	FINITE LIFE		>1000
1N-0N	2	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18N-17N	19	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
13N-35N	27	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
1S-0S	178	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18S-17S	194	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
13S-35S	203	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
2N-1N	241	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15N-18N	253	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18N-19N	254	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
6N-5N	320	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
6S-5S	324	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
10N-9N	327	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
10S-9S	331	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
2S-1S	569	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15S-18S	581	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
18S-19S	582	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15N-16N	14	Fatigue possible	INFINITE LIFE	INFINITE LIFE		
15S-16S	189	Fatigue possible	INFINITE LIFE	INFINITE LIFE		

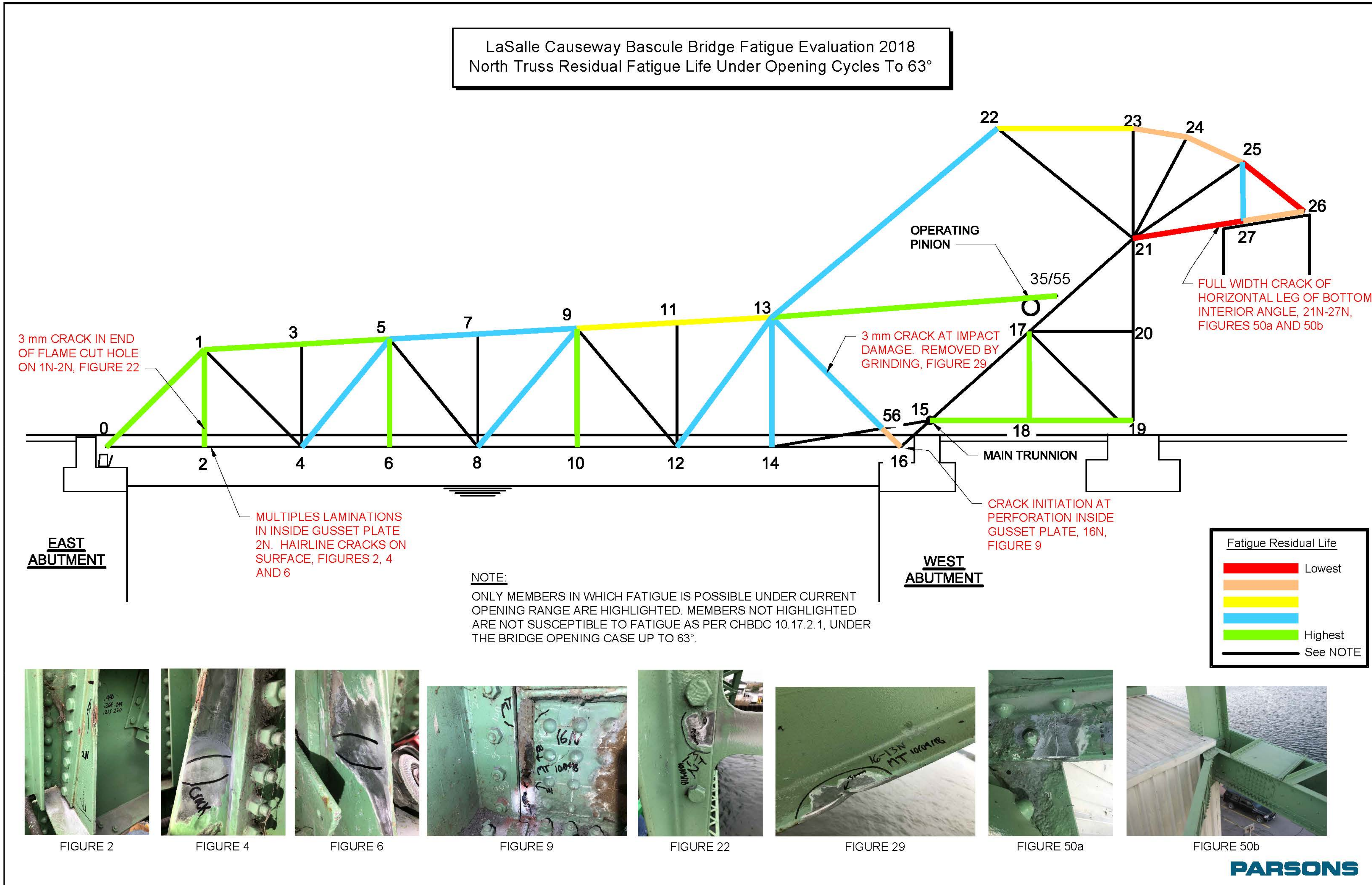


Figure 31: Fatigue Residual Life Under Bridge Opening Cycle – North Truss

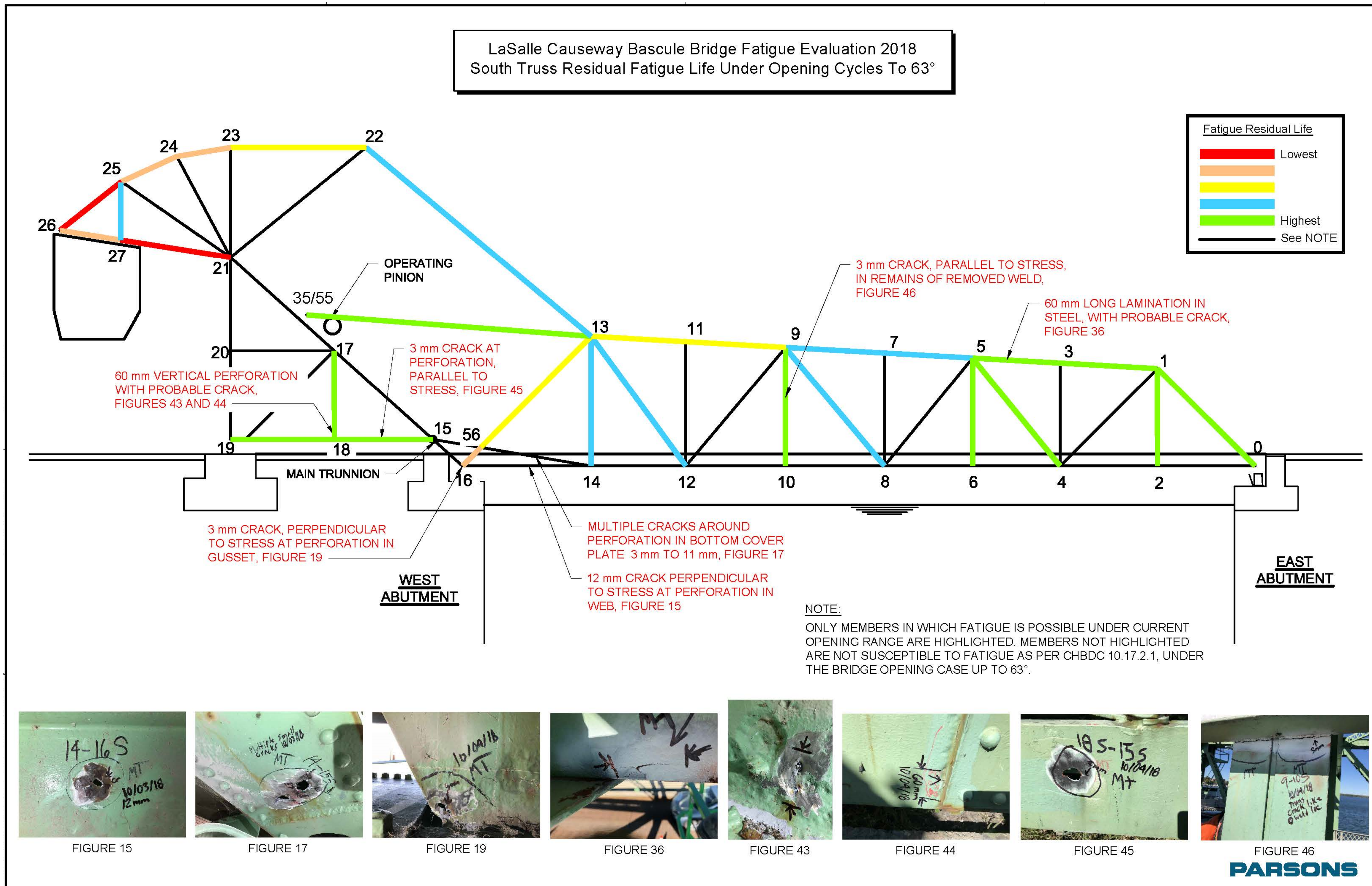


Figure 32: Fatigue Residual Life Under Bridge Opening Cycle – South Truss

5.4.7 ANALYSIS OF RESULTS OF EVALUATION FOR BRIDGE OPENING CYCLES

The results show that the most vulnerable members are located in the counterweight truss. Members 25N-26N, 25S-26S, 21N-27N, 21S-27S are the members with the shortest residual life. When conservative assumptions are made about the net section of members 26S-27S and 26N-27N, they have residual life comparable to member 25-26 and 21-27[3].Those members would have reached their theoretical fatigue life if detail Category D is considered, for both Upper and Lower bound scenarios of residual life. Since those members have internal redundancy, the rationale of AASHTO MBE to use Category C is applicable. With Category C and for the upper bound of residual fatigue life, those members would have a residual life in excess of 150 years. However, they would have reached the end of the theoretical fatigue life 16 to 37 years ago in the conservative lower bound scenario for residual fatigue life.

Since no cracking at the net section of counterweight members has been found to date, it seems that the upper bound scenario is the more realistic scenario. With Category C, all members have a significant residual life. However, cracking was found in one of the counterweight members in 2018. As described earlier, one angle of 21N-27N was found to be cracked, but not at the net section. The crack initiation seems to have been cause by either the increase of stress range at a location were section loss occurred or from a sharp indentation created by corrosion. Removal of the cracked angle and laboratory analysis is required to confirm this assumption, however, based on the observations made at the crack location, it seems to be the most probable cause.

The corrosion and section loss occur during the service life of the bridge and the number of cycles seen by the corroded section is less than the total number of cycles seen by the bridge. Corrosion worsens the Fatigue Detail Category C, or probably even D, since the uncorroded members have resisted many cycles without any sign of distress. This underlines the importance of preventing further section loss in fatigue vulnerable members. Recommendations to address the section loss and prevent future section loss are presented in the next sections.

5.5 Uncertainties Associated with Fatigue Evaluation

Many uncertainties are related to the fatigue evaluation for the vehicular loads and for the opening cycle. This is mostly due to the long life of the bridge and the absence of records for many years. The residual Fatigue life of the members could have been influenced by extreme events such as accidental impact load during bridge operation (at least one known “dropped bridge” event), special permit vehicular loads, high winds on the opened bridge, etc. Moreover, only main stresses were considered in the theoretical evaluation. Uneven stress share between member component and secondary bending were not considered and can increase local stress ranges. Thus, the presented theoretical results must be taken with caution and seen as a relative fatigue vulnerability analysis.

5.6 Summary of Fatigue Evaluation Recommendations

So far, most of crack founds originated from corroded or perforated areas. Given the fact that corrosion and perforations were present only for a part of the bridge service life and that only one crack has been found at net

[3] In the result tables, the assumption from the 2017 Structural Evaluation were used for the effective net section of 26S-27S and 26N-27N. Those assumptions are less conservative than the one made in the 1997 Stringer Report. The difference in the assumptions is related to the effective section area of 26-27. Elements were added to 26-27 after the construction. Such elements would be mobilized at ULS, but to calculate the resistance of 26-27 to FLS, it is conservative to use the original section, since cumulative damage has occurred in the original elements before the addition of new steel elements. With conservative assumptions for net sections, upper bound results for category C and D residual life are 211 years and -1 years for 26N-27N and they are 214 years and 0 years for 26S-27S. Lower bound results for category C and D residual life are -23 years and -62 years for 26N-27N; -23 years and -62 years for 26S-27S

section of rivetted connection (13N-14N, 2016)[4], it is possible to assume that, for the LaSalle Causeway Bascule Bridge, sharp indentations in corroded or perforated areas behave as worse fatigue detail than net section of riveted connection. Those indentations allowed cracks to initiate and propagate under less loading cycles than fatigue details associated with rivetted connections. As corrosion progress and more loading cycles are applied to the bridge, corrosion cracking is likely to increase in severity and frequency. If is therefore of paramount importance to limit the progression of corrosion and improve the profile of perforated or heavily corroded areas to obtain surface less favorable to crack initiation, especially on most vulnerable members. As some members reach the end of their theoretical fatigue life for Category D or even for Category C, cracking at net section of riveted connection might appear in member with the highest stress ranges. Regular fatigue inspection targeting the most vulnerable members would be beneficial to mitigate the risk associated with fatigue cracking. Fatigue Evaluation recommendations are summarized in Table 7.

Table 7: Summary of Fatigue Evaluation recommendations

Fatigue Evaluation Recommendations
Prevent progression of corrosion in steel members, in particular in members with high stress range from vehicular traffic or from bridge opening.
Perform biannual Fatigue Inspection targeting the most fatigue vulnerable members to allow early detection of fatigue cracks. Special attention should be given to net section of riveted connections and at sharp indentations in corroded of perforated areas.
Perform Preventive Maintenance to remove sharp indentations created by corrosion and repair perforations. Grind sharp indentations and perimeter of perforation. Add slip critical bolted plates when necessary. Priority should be given to the most fatigue vulnerable members.
Keep bridge paint in high condition to prevent further section loss and associated increase of fatigue stress range
Keep bridge paint in high condition to ease early crack detection.
Coordinate inspections and bridge cleaning to ease early crack detection

5.7 Fatigue Vulnerability Assessment Tool

In order to better classify the main truss members by relative fatigue vulnerability, other factors could be considered in addition to the theoretical residual life under bridge opening and traffic loading. Factors such as member structural redundancy, eccentricities in connection, previous cracking at similar locations, stress range or Miner's factor C (spent fatigue life fraction under cumulative effects of bridge opening and traffic loading), secondary stresses, equivalent fatigue detail category for perforations and corrosion, unequal load sharing between component in the member, etc. With those factors, a Vulnerability Matrix could be developed. Each member would get a score for every factor considered and a global score would be calculated. From this score, members could be classified by relative Fatigue Vulnerability and Fatigue Inspection Frequencies could be attributed to each member. Table 8 shows an example of a Vulnerability Matrix for the main members of the LaSalle Causeway Bascule Bridge.

[4] Cracks on 13N-14N might also have been caused by unknown factors or extreme events, such as a potential vehicular impact on the guardrails, transmitted to the vertical.

Table 8: Example of a Possible Fatigue Vulnerability Matrix

Class of Element	Type of Element[1]	Node/ member	Fatigue type[2]/ Fat. Detail Cat	Presumed cause of cracking	Vunerability								
					1-Member Structural Redundancy	2-Stress Range	3-Member Internal Redundancy	4-Presence cracking	5-Excentricities/ shear lag	6-Secondary Stresses	7-Fatigue Category Detail/Equivalent damage category	Score	Member Vulerability
Main truss top chord	Member	0N-1N			10	2	8	0	4	10	4	38	moderate
	Member	1N-3N			10	2	8	0	4	10	4	38	moderate
	Member	3N-5N			10	2	8	0	4	10	4	38	moderate
	Member	5N-7N			10	2	8	0	4	10	4	38	moderate
	Member	7N-9N			10	2	8	0	4	10	4	38	moderate
	Member	9N-11N			10	2	8	0	4	10	4	38	moderate
	Member	11N-13N			10	2	8	0	4	10	4	38	moderate
	Member	0S-1S			10	2	8	0	4	10	4	38	moderate
	Member	1S-3S			10	2	8	0	4	10	4	38	moderate
	Member	3S-5S		LIF[9]	10	2	8	8	4	10	10	52	very high
	Member	5S-7S			10	2	8	0	4	10	4	38	moderate
	Member	7S-9S			10	2	8	0	4	10	4	38	moderate
	Member	9S-11S			10	2	8	0	4	10	4	38	moderate
	Member	11S-13S			10	2	8	0	4	10	4	38	moderate
	Gusset	13S			8	2	10	0	0	6	4	30	moderate
Main truss verticals	Member	1N-2N		WIF[9]	10	2	4	8	6	10	4	44	high
	Gusset	1N-2N		LIF/WIF[9]	10	2	4	8	6	10	4	44	high
	Member	5N-6N			10	2	4	0	6	10	4	36	moderate
	Member	9N-10N			10	2	4	0	6	10	4	36	moderate
	Member	13N-14N		LOIFC	10	2	4	10	6	10	4	46	high
	Member	1S-2S			10	2	4	0	6	10	4	36	moderate
	Member	5S-6S			10	2	4	0	6	10	4	36	moderate
	Member	9S-10S		WIF[9]	10	2	4	8	6	10	4	44	high
	Member	13S-14S			10	2	4	0	6	10	10	42	high

6.0 References

- 1997 – David C. Stringer - LaSalle Causeway Bascule Bridge Fatigue Evaluation,
- 1998 – John W. Fisher, Geoffrey L. Kulak, Ian F. C. Smith, A Fatigue Primer for Structural Engineers, National Steel Bridge Alliance.
- 1998 – Theodore V. Galambos – Guide to Stability Design Criteria to Metal Structures (GSDCMS), 5th Edition
- 2003 – André Picard, Denis Beaulieu - Calcul des charpentes d'acier, ICCA/CISC
- 2005 - Andy Huctwith -Kingston Bascule Bridge Fatigue Review and Rehabilitation of Counterweight Members (updated after construction), MRC
- 2012 - AASHTO LRFD Bridge Design Specifications 6th Ed. Part I Sections 1 - 6
- 2014 – Construction Challenges During Heel Trunnion Replacement for an Historic Strauss Bascule (Cherry Street Bridge Toronto), John Williams SBE, Ian Funkenhauser Facca, Heavy Moveable Structures 15th Biennial Symposium.
- 2014 – Canadian Standards Association (CSA) - Canadian Highway Bridge Design Code (CHBDC) S6-14, December 2014.
- 2015 – AASHTO LFRD Moveable Highway Bridge Design Specifications (2nd Edition) with 2008, 2010, 2011, 2012, 2014, and 2015 Interim Revisions.
- 2016 - Computers & Structures, Inc. (CSI) - Analysis Reference Manual for SAP2000, ETABS, SAFE, and CSiBridge - Berkeley, California, USA, July 2016
- 2017 – Maurice Mansfield, Amir Tehranian – LaSalle Causeway Bascule Bridge 2017 Structural Evaluation Report (and associated CSI Bridge models and spreadsheets), Parsons.
- 2017 - Ministère des Transport du Québec Bridge Evaluation Manual
- 2018 – AASHTO Manual for Bridge Evaluation (3rd Edition)
- 2018 - PSPC Traffic and Bascule Bridge Openings data (via emails)

7.0 Closure

We trust that this report is adequate for your present requirements. If you have any comments or questions, please contact the undersigned.

Yours truly,

PARSONS INC.

Report and Calculations prepared by:

2019-03-04

Jean-Bernard Charron, P.Eng., ing.

Report Reviewed by:

Adriano DiRienzo, P.Eng.

Calculations prepared by:

Kevin Serre, ing.

Calculations Reviewed by:

2019-03-06

Dennis Bascope, P.Eng., ing.