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BC Ministry of Transportation and Infrastructure

Old Spences Bridge

2012 Inspection Report

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Rev 1 – Minor edits to text and sentence structure

Executive Summary

The Old Spences Bridge was constructed in 1931 and provided Highway 1 with a crossing over the Thompson River in the Community of Spences Bridge, BC. In 1962, a new bridge was constructed approximately 900 m downstream.

Buckland & Taylor performed a detailed inspection of Old Spences Bridge and carried out a load rating in 2009. The BC MoT performed further inspections of the structure in 2010 and 2011 and subsequently requested that B&T update the 2009 load evaluation, conceptual rehabilitation and cost estimates based on the recent inspection findings. It was also requested that the updated evaluation consider a more strict criteria that included a wind and seismic evaluation. These updated recommendations were included in a November 2012 report that contained a requirement that a detailed inspection of the bridge be performed before the end of 2012 if the bridge was to continue to accommodate traffic beyond 2012. At the Ministry's request, a detailed inspection was performed by B&T on the Old Spences Bridge between 2012 December 10 and December 14.

Buckland & Taylor's 2012 inspection of the Old Spences Bridge included an inspection of the overall condition of the bridge but was focused on areas identified in the 2012 Updated Load Evaluation Report as areas of concern. While section loss were observed in the areas of concern, the majority of locations were found to be within acceptable tolerances based on calculated capacities for continued safe use of the structure in the short term. However, if the bridge is to remain in service beyond the end of 2013 the following recommendations are made:

- Repair deteriorated deck stringers;
- Repair deteriorated sidewalk stringers;
- Repair floorbeam flanges;
- Repair floorbeam webs;
- Replace sway bracing gusset plates;
- Repair the concrete piers;
- Extend deck drains;
- Replace bottom chord batten plates on an as needed basis and;
- Replace main truss gusset plates.

Of the above noted items, the following are considered as having the highest priority:

- Replace the sway bracing gusset plate in Span 4; and,
- Repair the webs of three floorbeams in Span 4.

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1 Introduction

The Old Spences Bridge, as shown in Figure 1, was constructed in 1931 and provided Highway 1 with a crossing over the Thompson River in the Community of Spences Bridge, BC. The structure's north end also crosses a set of CN Rail tracks. The structure remained as the main crossing in the area until 1962 when a new bridge was constructed approximately 900 m downstream.

The Old Spences Bridge is owned and maintained by the British Columbia Ministry of Transportation and Infrastructure (BC MoT).

Figure 1: General View of the Old Spences Bridge from the South Abutment

The Old Spences Bridge is a single-lane bridge composed of five truss spans and two girder spans. The truss spans vary in length with one span of 21.0 m (69 ft), two spans of 27.7 m (91 ft) and two spans of 65.8 m (216 ft). The girder spans are at the north end of the bridge and are 11.3 m (37 ft) and 12.2 m (40 ft) long, making the total length of the bridge 231.6 m (760 ft). Six concrete piers and two concrete abutments support the bridge. An elevation, plan and typical sections of the bridge are shown in Figure 2 and Figure 3, and a general arrangement drawing is included in Appendix A.

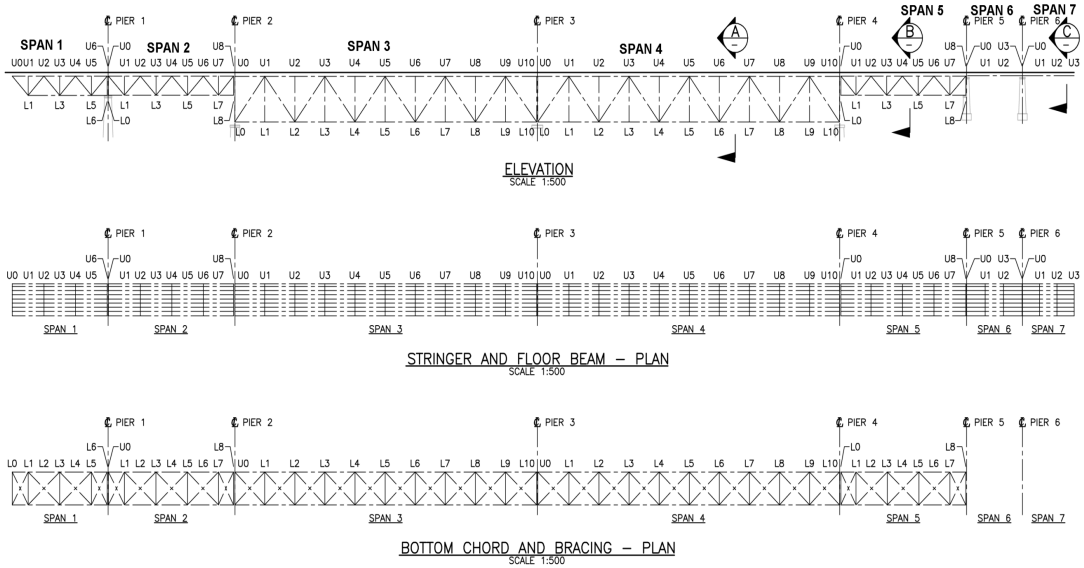


Figure 2: Old Spences Bridge - Elevation and Plan Views

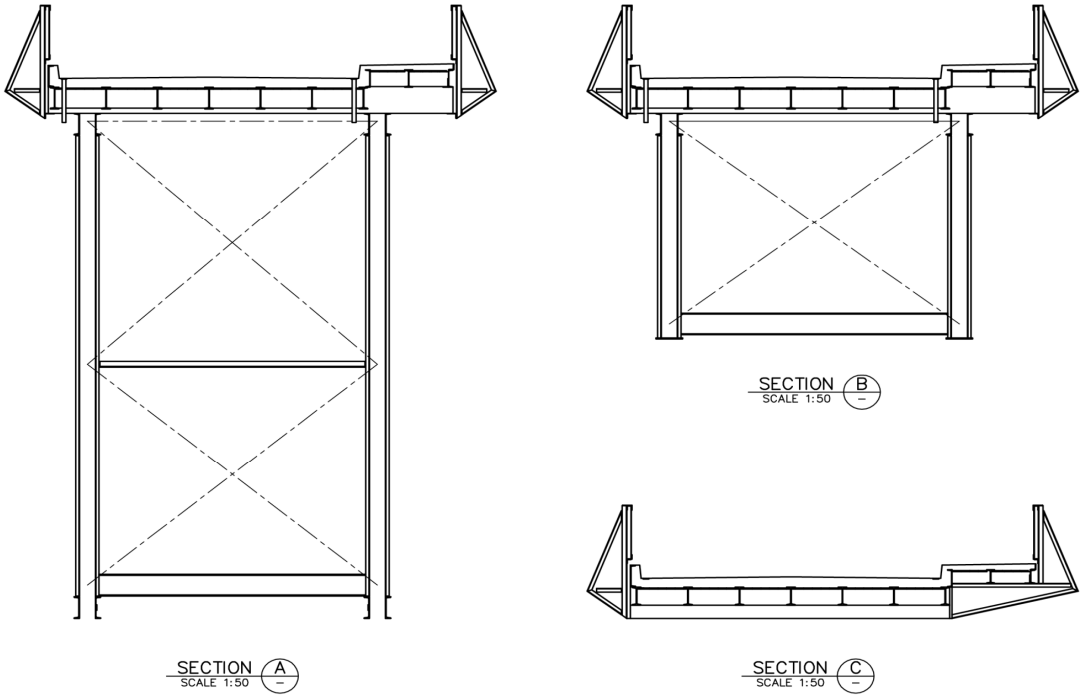


Figure 3: Old Spences Bridge - Typical Cross Sections

1.1 Condition Assessment Background

Following a routine inspection performed by BC MoT in 2002, the bridge was posted with a 25 tonne load limit restriction due to observed corrosion damage in structural members. During a 2008 visual inspection by BC MoT, significant deterioration, corrosion and perforations were identified in heavier structural components. Based on these observations the bridge was closed to all vehicular traffic in 2009 in order to ensure public safety.

Subsequent to closing the crossing, BC MoT retained Buckland & Taylor (B&T) to carry out a detailed inspection and load capacity evaluation of the structure. As part of the assignment, B&T was also tasked with developing conceptual rehabilitation options and cost estimates to restore the bridge to a range of acceptable levels of reliability. These evaluations and recommendations varied from a 'do nothing' approach to a complete bridge replacement, and were provided in the two reports detailed below:

- B&T 2009 Inspection Report: B&T Report No. 1884-RPT-SPE-001-0, “Old Spences Bridge No. 2411 – Inspection Report”. The report presented maintenance, rehabilitation and evaluation items based on observations made during the 2009 detailed inspection; and
- B&T 2009 Load Evaluation Report: B&T Report No. 1884-RPT-SPE-002-2, “Old Spences Bridge No. 2411 – Load Capacity Evaluation & Rehabilitation Options”. The report summarized the findings of the load evaluation of the bridge, made recommendations regarding conceptual rehabilitation options, and summarized high-level cost estimates for a variety of live load models and service life options. The report also found that the bridge could be re-opened to vehicular traffic, subject to a posted load restriction of 5 tonnes.

Based upon B&T's evaluation results, the bridge was re-opened with a posted load restriction of 5 tonnes. It was also recommended in the evaluation report that annual detailed inspections be performed if the bridge was re-opened. BC MoT has performed these inspections and prepared the following reports:

- BC MoT 2010 Inspection Report: "Old Spences Bridge No. 2411 - Inspection Report" (for inspections performed in early November 2010); and
- BC MoT 2011 Inspection Report: "Old Spences Bridge No. 2411 - Inspection Report" (for inspections performed in mid July 2011).

Following the 2011 BC MoT inspection, B&T was requested by BC MoT to update the 2009 load evaluation, conceptual rehabilitation and cost estimate assessment based on the Ministry's recent inspection findings, with consideration given to some changes in scope as listed below. For the purposes of confirming that the bridge condition was in general conformance with that reported by BC MoT in 2011, a brief site assessment was performed on 2012 March 08 by B&T engineers. The site visit was not a detailed inspection, and access was limited to observing the bridge from the deck and from the shores. Subsequent to the site visit, updated recommendations were provided by B&T in the following report:

- B&T 2012 Updated Load Evaluation Report: B&T Report No. 1976-RPT-GEN-001-1, "Old Spences Bridge No. 2411 – Update of Load Capacity Evaluation and Rehabilitation Options". The report presented updated information from the 2009 Load Evaluation Report based on the findings of the 2010 and 2011 BC MoT inspection reports, with the following changes in scope:
  i. inclusion of a wind and seismic evaluation (not previously considered);
  ii. exclusion of the CL1-625 live load model due to the numerous and excessive overstresses throughout the structure that would have led to uneconomical strengthening schemes; and
  iii. exclusion of the 25 tonne live load model due to extent of required rehabilitation and strengthening.

While the brief site visit performed by B&T prior to the production of the 2012 Updated Load Evaluation Report confirmed that the general condition of the bridge was consistent with that reported in the 2011 BC MoT Inspection Report, some additional observations were noted which were deemed to require further assessment.

1.2 Current Assignment

At BC MoT's request, a selective detailed inspection was performed by B&T on the Old Spences Bridge between 2012 December 10 and December 14 by Brandon McGinnis, P.E. and Todd McCrimmon, EIT. This report summarizes the observed findings during the hands-on inspection of the bridge.

1.2.1 Inspection Scope

The aim of the 2012 inspection was to review the condition of key components of the structure in order to provide a level of confidence to the owner for keeping the bridge open until 2013 December 31. The key components were identified in B&T's 2012 Updated Load Evaluation Report (reference B&T report 1976-RPT-GEN-001-1 "Update of Load Capacity Evaluations and Rehabilitation Options") as requiring further inspection in order to confirm assumptions made regarding their capacity to carry the required loads in the short and long term. To this end, the work consisted of inspections of steel truss spans and the concrete piers but did not include an inspection of the two girder spans, the concrete deck or the railing. The focal points of the inspection are listed in Section 1.2.1.1 below.

1.2.1.1 Detailed Inspection Items

While all of the bridge members were reviewed during the inspection, the focus of the inspection was on the members identified in the 2012 Load Evaluation Report as having a Live Load Capacity Factor (LLCF) equal to 1.5 or less. These components are identified in Table 1 and are also shown in Figure 4 and Figure 5 as having an LLCF between 1.0 and 1.5 (shown in green), or an LLCF less than 1.0 (shown in red).

Table 1: Focus Areas of the 2012 Inspection

Item	Focus Areas
Stringers	• Areas of section loss and heavy corrosion on top flange.
Floorbeams	• Areas of section loss and heavy corrosion on top flange. • Areas of section loss in the web above bearing supports on the top chord.
Main Truss Members	• Diagonals and connections as identified in green; refer to Figure 4. • Secondary Batten plates on top and bottom chords (areas of section loss). • Gusset plates to vertical members (section loss). • Gusset plates to diagonals (areas of section loss along the length of the gusset plate immediately above the bottom chord channel).
Lateral Load Carrying Members	• Members with section loss and advanced corrosion, refer to Figure 5. • Section loss in gusset plates.
Bearings	• Number of rivets in Pier 2 bearings, refer to Figure 6. • Condition of anchor bolts.
Concrete Piers	• Horizontal cracks.

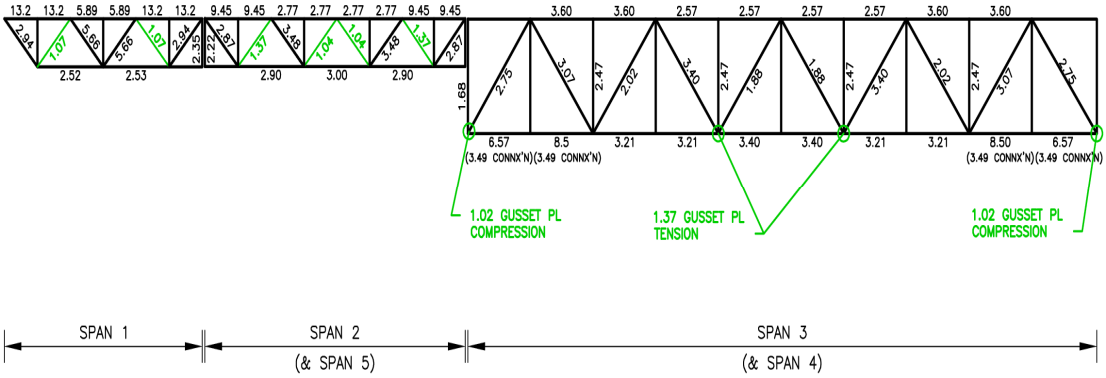


Figure 4: Truss Member LLCF's and Connection Focus Areas

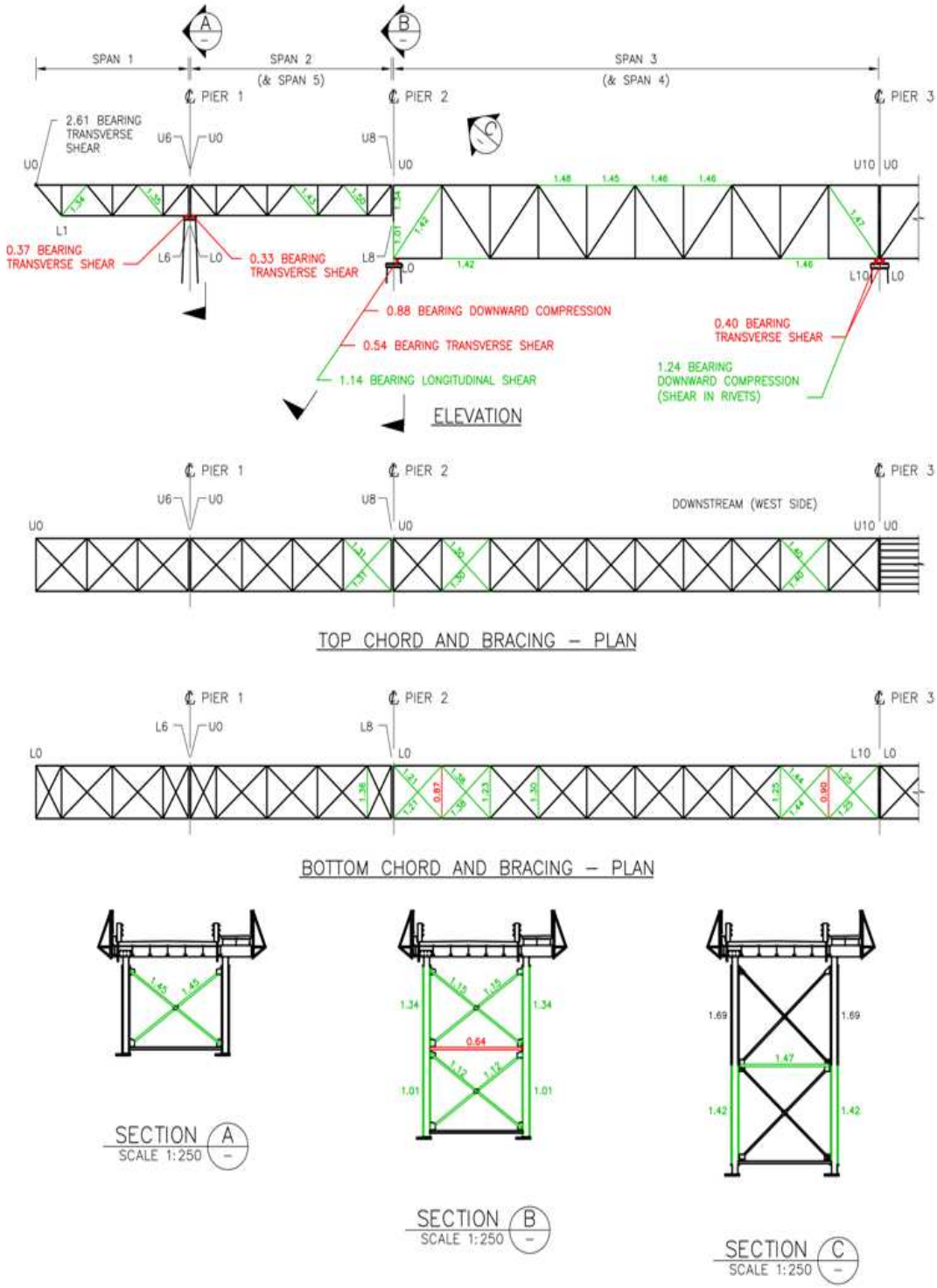


Figure 5: Truss Member LLCF's for the Lateral Load Carrying Focus Areas

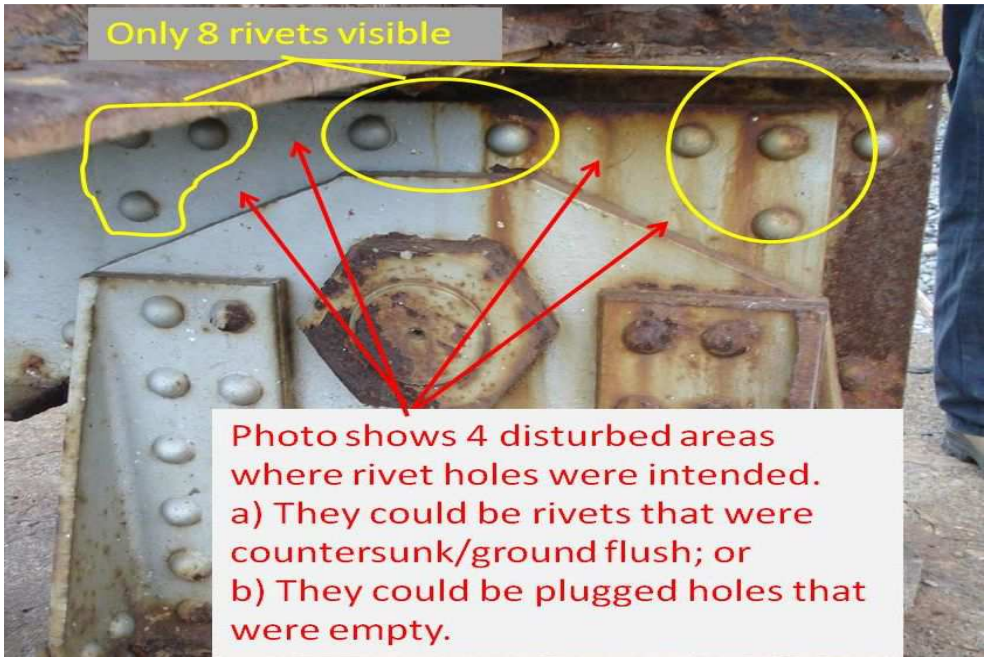


Figure 6: Pier 2 Bearing Focus Area (Identified in the Load Evaluation Report)

1.2.2 Inspection Procedure

Given the 5 tonne load restriction currently imposed on the Old Spences Bridge it was not possible to inspect the structure using an under-bridge inspection vehicle. As a result, and similar to what was successfully adopted in the 2009 detailed inspection, the inspection team utilized swing stages and bridging units to provide the required access, refer to Figure 7 and Figure 8. The swing stages were supported from a scaffolding system assembled on casters on the bridge deck which enabled the unit to be positioned at any location along the deck, refer to Figure 9.

Figure 7: Swing Stage Access

Figure 8: Bridging Plank between Swing Stages

The North and South Abutments, and portions of Piers 5, 6, and Span 1 were accessed from the ground given the access provided by the local topography.

Figure 9: Scaffolding System Used above Deck to Support the Swing Stages

2 Bridge Component Details

The framing of the truss spans consists of top chords, top chord lateral bracing, verticals, diagonals, bottom chords, bottom chord lateral bracing and transverse sway bracing. The deck framing system consists of longitudinal stringers supported on transverse floorbeams, which bear on the top chord of the truss spans.

Each girder span consists of longitudinal stringers supported on two transverse floorbeams, which frame into two longitudinal edge girders. The edge girders are supported on concrete piers and abutments.

The bridge has been assembled using rivets although areas in which repairs have been made use high strength bolts.

The main bridge components are identified in Figure 10 to Figure 13, and are described in more detail in the subsections that follow.

Any dimensions given in this section refer to the size of the member that was expected at installation.

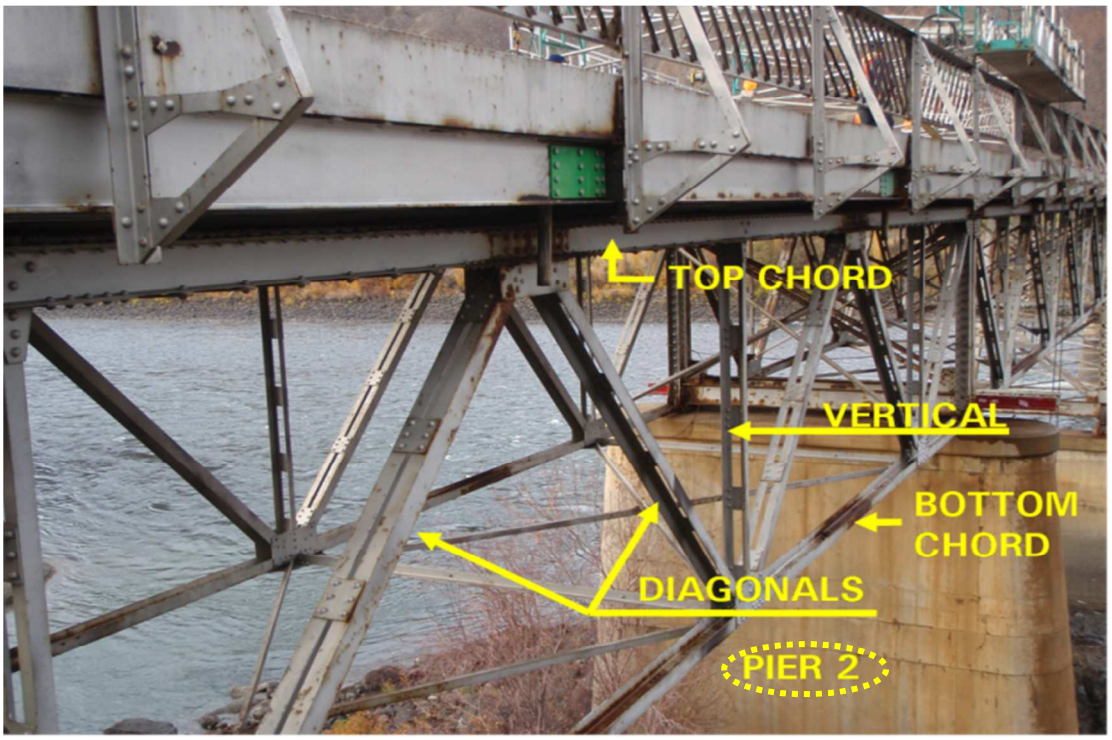


Figure 10: View of Typical Truss Span Showing Vertical Load Carrying Members

Figure 11: View of Typical Truss Span Showing Lateral Load Carrying Members

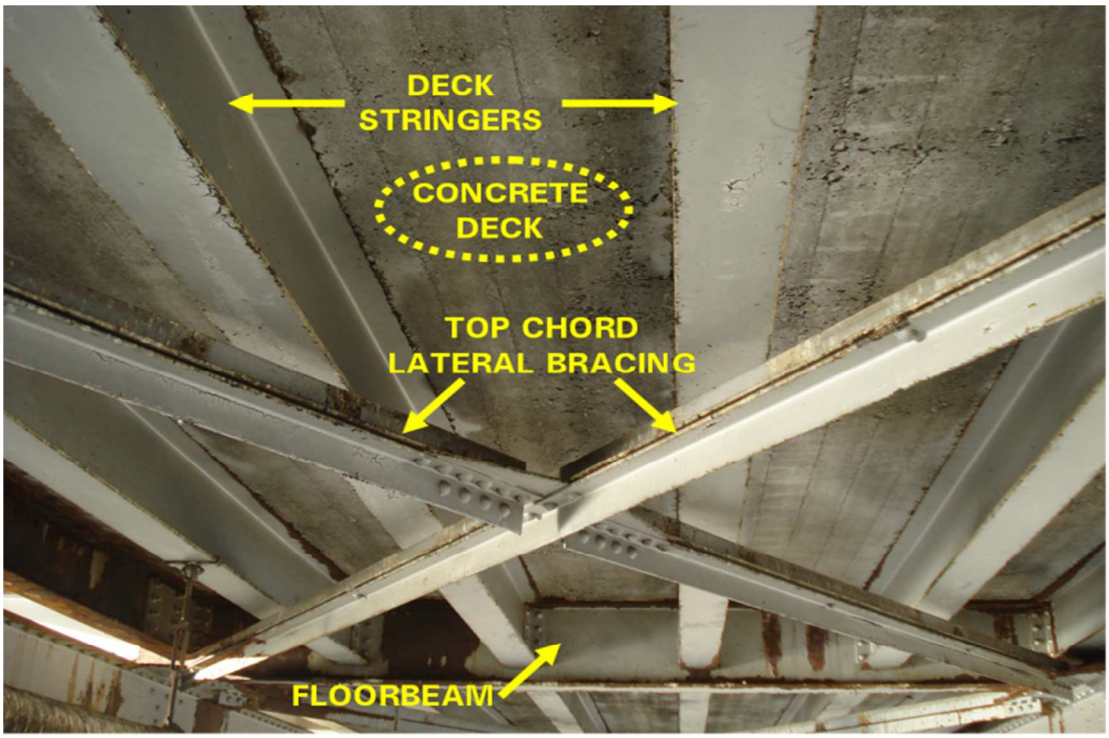


Figure 12: View of Typical Floor System in Truss Spans

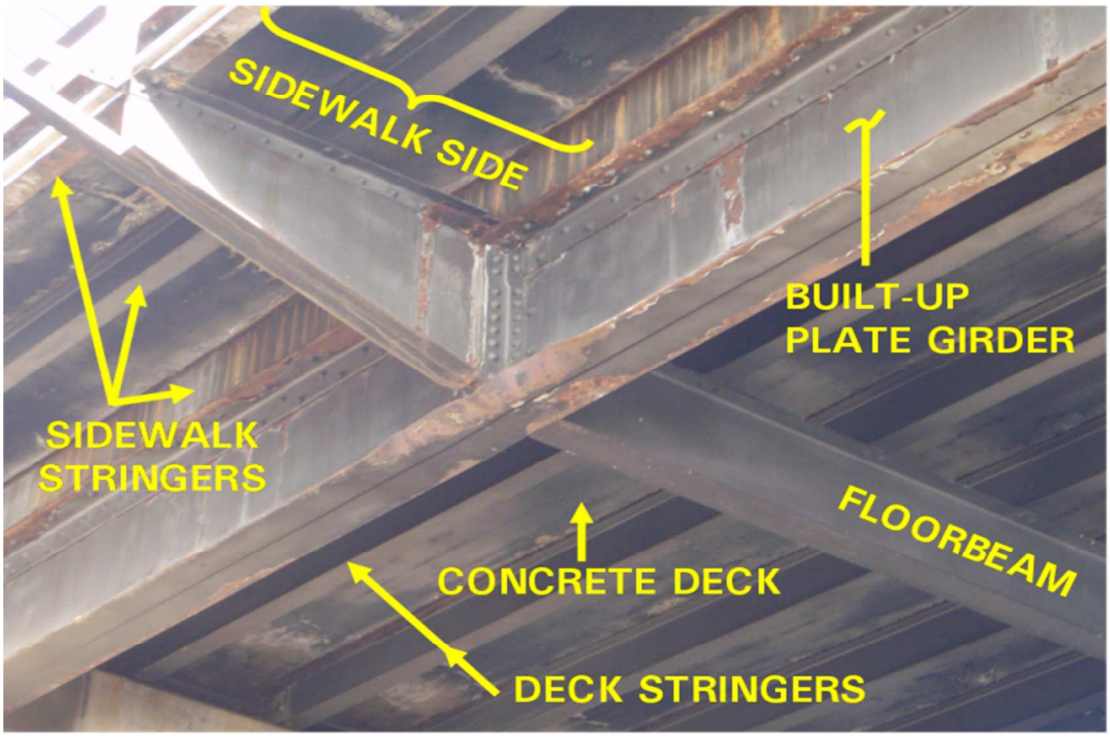


Figure 13: View of Typical Floor System in Girder Spans

2.1 Top Chord

The top chords of the truss spans are formed from back-to-back rolled channels that are connected along the top flange using a combination of batten plates and continuous cover plates. Along the bottom flange, the channels are connected using lacing bars. In Spans 1, 2 and 5 the channels are 203 mm (8”) deep while in Spans 3 and 4 they are 380 mm (15”) deep.

2.2 Bottom Chord

Unlike the top chords, the type of members making up the bottom chords differs between the longer and shorter spans. In the longer spans, Spans 3 and 4, the bottom chord members are two back-to-back 380 mm (15”) deep channels connected by batten plates along the top and bottom flanges. However, in the shorter spans, Spans 1, 2 and 5, the bottom chords are formed by pairs of steel angles oriented toe-to-toe with the vertical leg extending upwards. The angles are connected with batten plates at approximately quarter points along their length.

2.3 Verticals

The vertical members throughout all of the truss spans are either formed from pairs of steel angles or pairs of steel channels. In the shorter spans, pairs of angles are used exclusively while steel channels are used in the longer spans where member demands are larger.

2.4 Diagonals

The diagonal members in the truss spans are similar to the vertical members with pairs of steel angles used in the shorter spans and pairs of steel channels used in the longer spans. However, the tension diagonals in Spans 3 and 4 are formed from four angles as opposed to the pair of angles used in the shorter spans. The four angles are arranged in a box pattern connected at intermediate points with batten plates. Batten plates are also used to provide intermediate connections between members.

2.5 Bottom Chord Lateral Bracing

The bottom chord lateral bracing in all of the truss spans comprises single steel angles as diagonal cross-bracing and pairs of angles as transverse struts. The pairs of angles are oriented back-to-back with vertical legs oriented upwards. At the bearing locations the transverse strut is a rolled I-shape girder in place of the pairs of angles. This girder serves as a jacking beam for bearing replacement and may provide a means of balancing loads between the bearings.

The cross-bracing members frame into gusset plates that are riveted to the underside of the bottom flange of the bottom chord in the case of the shorter spans, and to the top flange of the bottom chord in the case of the longer spans. A gusset plate is also located at the intersection of the two cross brace angles to provide a mid-length connection.

2.6 Top Chord Lateral Bracing

Similar to the bottom chord lateral bracing, the top chord lateral bracing is generally formed with single angles as diagonal cross-bracing members. The only exception is in Spans 3 and 4, where some of the diagonal cross-bracing is made up of back-to back angles. Unlike the bottom lateral bracing however there are no transverse struts. These struts are replaced with the floorbeams that support the concrete deck.

The cross-bracing members are connected to gusset plates at each end of the member. These gusset plates are located between the top chord flange and the bottom flange of the floorbeams. A gusset plate is also located at the intersection of the two cross brace angles to provide a mid-length connection.

2.7 Sway Bracing

Sway bracing is provided between the east and west trusses at end points and intermediate points. The framing of the bracing is either single or double angles connected at their intersection point and at their endpoints to the east and west trusses.

In Spans 3 and 4, the sway bracing is located at Panel Points 0, 2, 4, 6, 8 and 10. There is also a set of inclined sway bracing in the end bays of the truss where the top chord frames into the bearing point at the pier (eg. Panel Points L0 to U1). Because the truss is fairly deep in Spans 3 and 4, the sway bracing is divided into two bays over the depth of the truss. The transverse strut at mid-height of the sway bracing is a built up member consisting of two or four angles.

In the shorter spans, the sway bracing is oriented on a slope and is connected to the truss diagonals. In Span 1, sway bracing is located between Panel Points 0 and 1 and between Panel Points 5 and 6. In Spans 2 and 5, sway bracing is located between Panel Points 0 - 1, 2 - 3, 5 - 6 and 7 - 8.

2.8 Deck Components

A 150 mm (6”) concrete deck supported on longitudinal stringers, which are in turn supported on transverse floorbeams, makes up the deck system. The concrete deck is believed to be the original cast-in-place bridge deck. It appears that the deck was cast as individual panels between adjacent floorbeams resulting in joints in the concrete at each floorbeam location. The design drawings show a single mat with two layers of reinforcing located 37 mm (1.5”) from the underside of the deck.

The deck stringers in the truss spans are 14" C.B. 36 # rolled steel I-sections (as described in 1928) which consist of a 356 mm (14") deep beam with 172 mm (6.75") wide flanges. The thickness of each flange is around 12 mm (0.50") and has a web thickness of 7 mm (0.28"). The floorbeams in the truss spans are 18" C.B. 58 # rolled steel I-sections which consist of a 464 mm (18") deep beam with 192 mm (7.56") wide flanges. The thickness of each flange is 17 mm (0.67") and has a web thickness of 10 mm (0.39").

There is a 1220 mm (4 ft) wide sidewalk on the west side of the bridge that extends beyond the west truss. This sidewalk is supported on three longitudinal stringers that are also connected to the transverse floorbeams.

2.9 Girder Spans

The two girder spans, Spans 6 and 7, are located at the north end of the bridge and measure 12.2 m and 11.3 m (40 ft and 37 ft), respectively. The south span, Span 6, crosses over an active CN Rail line containing two rail tracks. Both girder spans have the same framing arrangement with two 710 mm (28”) deep built-up plate girders supporting the spans. The plate girders are constructed with four angles riveted to a web plate. Each span has five longitudinal deck stringers that are continuous along the span. The stringers have bearing plates at each end where they rest on concrete pedestals. Intermediate support is provided at the third points where the stringers bear on transverse floorbeams. The floorbeams are connected to the edge girder with a web-to-web connection. Both the stringers and the floorbeams are rolled I-shaped sections.

2.10 Bearings

Each of the seven spans is supported at one end on fixed bearings and on the other end by sliding bearings. The fixed bearings are defined as bearings that restrict longitudinal, transverse and vertical translation while permitting rotation about the transverse axis. The sliding bearings are defined as bearings that permit translation in the longitudinal direction and rotation about the transverse axis. All other rotations and translations are restricted.

3 Inspection Findings

The findings of this inspection are presented in the following sections. As noted in Section 1.2.1, the focus of the inspection was on the members identified in the 2012 Load Evaluation Report as having a Live Load Capacity Factor (LLCF) equal to 1.5 or less, however all of the bridge members were reviewed.

3.1 Stringers

The longitudinal stringer system has been broken into two categories; deck stringers (DS) and sidewalk stringers (SS); refer to Figure 14 below.

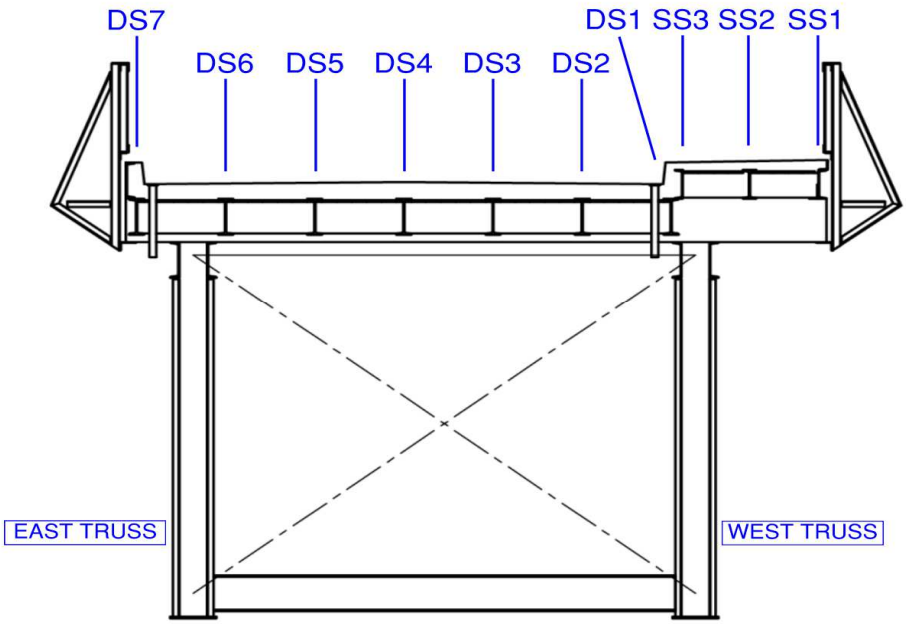


Figure 14: Deck Stringer Arrangement in the Truss Spans

3.1.1 Deck Stringers

For the purpose of reporting the inspection findings, the deck stringers have been further divided into two sub categories: exterior and interior stringers. The exterior stringers, DS1 and DS7 were typically found to have more significant deterioration than the interior stringers, DS2-DS6. This is illustrated in Figure 15, and is likely due to their increased exposure at each side of the bridge, and in the case of DS1, being located below the curb.

Figure 15: Variation in the Deterioration between a DS6 Interior Stringer and a DS7 Exterior Stringer (right) near FB10 in Span 4

3.1.1.1 Exterior Deck Stringers

Numerous widespread areas of coating failure and surface corrosion were observed along many of the exterior stringers. As typical with all the stringers, the most notable deterioration was observed at the ends of the members; within approximately 100-200 mm (4-8”) of the connection with the floorbeams. This deterioration has resulted in localized areas of section loss, including small perforations in the web on some members; refer to example shown in Figure 16 and Figure 17. Significant amounts of corrosion product build-up and associated section loss was also identified on the underside of the top flange and on the top and bottom surfaces of the bottom flange.

Figure 16: Inside Face of Exterior Deck Stringer DS7 at FB7 in Span 4

Figure 17: Outside Face of Exterior Deck Stringer DS7 at FB7 in Span 4

The corrosion and deterioration of the stringers has been identified in previous inspection reports and stringers with areas of section loss have mostly been repaired, although in at least two locations, DS1 in Span 1 between U0 and U2 (west) and DS1 in Span 3 between U5 and U6 (west), the stringers appear to have been marked for repair due to section loss in the web. Other observations of section loss include:

- Deck Stringer DS7 at Floorbeam 7 in Span 4, shown in Figure 16 and Figure 17, has a 45 mm x 20 mm web perforation at the floorbeam connection;
- Deck Stringer DS7 at Floorbeam 8 in Span 4 has a 20 mm x 20 mm web perforation at the floorbeam connection; and
- Deck Stringer DS7 at Floorbeam 5 in Span 3 has a 10 mm x 10 mm web perforation near the top flange at the floorbeam connection.

As outlined in the 2012 Updated Load Evaluation Report, the tolerable level of section loss is 50% of the web thickness over the entire web depth which is greater than the three locations identified and therefore repairs are not believed to be required prior to the end of 2013. However, if the bridge is to remain in service beyond the end of 2013 it is recommended that the stringers be repaired.

3.1.1.2 Interior Deck Stringers

The majority of interior deck stringers were observed to have some coating failure and surface corrosion on the webs and flanges but are otherwise in good condition. Generally, the deterioration has occurred within approximately 100-200 mm (4-8") of the member ends at the connection with the floorbeams, as shown in Figure 18. This location specific deterioration is likely the result of water ingress from failed joint sealer in the deck above the floorbeams. The observed deterioration to the interior deck stringers was not advanced to the point of warranting immediate repairs.

Figure 18: Interior Deck Stringers at the South Face of Floorbeam 5 in Span 3, Exhibiting Minor Surface Deterioration at the Connections

As noted in previous inspection reports, many of the deck stringers are no longer in contact with the underside of the concrete deck slab due to the formation of pack rust between the floorbeam top flange and the underside of the concrete deck. These expanding layers of corrosion have resulted in rust jacking of the deck such that there is a sizeable gap between the concrete deck and the longitudinal stringers in many locations (i.e. in several locations the deck is resting only on the floorbeams and not on the stringers), as shown in Figure 19 and Figure 20.

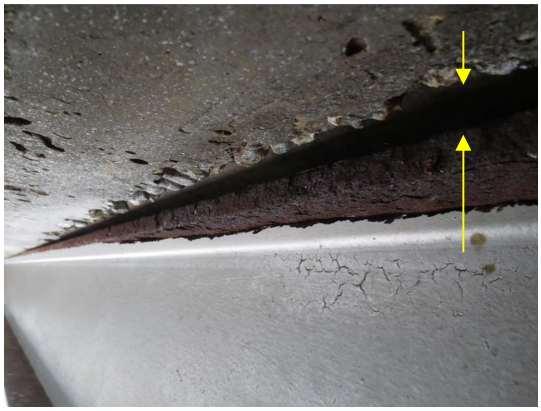


Figure 19: Gap above DS5 near FB10 in Span 4

Figure 20: Gap above DS4 near FB2 in Span 2

This effect reduces the dead load imposed on some stringers, while likely increasing it for those which remain in contact with the deck. This has also altered the behaviour of the concrete deck which was originally designed to span transversely, and is now having to span longitudinally; likely explaining some of the cracks on the top surface of the deck. This deficiency will have taken some time to fully develop and is not expected to become any greater an issue in the short term, therefore the rust jacking is not considered to be an immediate structural concern. However, it is recommended that grout be injected in the gaps between the deck and stringers in an effort to preserve the ride-ability and serviceability of the deck if the bridge is to remain in service beyond the end of 2013.

3.1.2 Deck Stringer Assessment Summary

The 2012 Updated Load Evaluation Report did not highlight the deck stringers as members of concern. The report also indicated that a 49% loss to the top flange cross section, and a 50% loss to the web thickness over the full depth of the member was tolerable for the 5-tonne and pedestrian-only loading scenarios.

During the 2012 inspection, no areas of section loss exceeding the tolerable limits were observed and therefore no recommendations for immediate repairs are recommended. For long term use of the bridge it will be necessary to stop the continued corrosion at the ends of the members and prevent it from recurring.

3.1.3 Sidewalk Stringers

Two of the three sidewalk stringers are I-sections while the third, SS3, is a channel section that also serves as the back side of the concrete curb for the sidewalk on the bridge deck.

It was observed during the 2012 inspection that the sidewalk curb is cracked and spalled in numerous locations which is allowing water to run down on to SS3. The west side of the web of SS3 was typically observed to be covered in surface corrosion over large areas along its length with isolated areas of minor to moderate section loss identified in the web near mid-span. More significant areas of section loss and perforations in the web were observed in multiple SS3 stringers at the connection at the floorbeams, refer to Figure 21.

Figure 21: Typical Location of Perforations in the Sidewalk Stringer, as shown for SS3 at FB0 in Span 4

A summary of the condition of the sidewalk stringer ends that have notable deterioration is provided in Table 2. It is believed that Sidewalk Stringer SS3 is primarily a stay-in-place form and is not relied upon for carrying loads. Therefore the localised areas of section loss are not believed to be a concern at this time. Pack rust was also commonly observed between the bottom flange of SS3 and the top

flange of DS1. This pack rust has caused localized deformations in the top flange of DS1 but is not believed to be a concern. The SS3 stringer is supported along its length by DS1 and the localized section loss or local deformations are not considered to be an immediate structural concern in the short term. It is recommended that stringer SS3 be replaced at the same time as any planned rehabilitation of the deck stringers.

Table 2: Condition of Sidewalk Stringers (SS3)

Stringer	Location	Issue
SS3 (U10-U9)	FB10 Span 4	Hole in the web half the width of the FB
SS3 (U1-U0)	FB0 Span 4	Hole in bottom of the web the entire width of the FB
SS3 (U10-U9)	FB10 Span 3	5 mm section loss at the bottom of the web the entire width of the FB
SS3 (U1-U0)	FB0 Span 1	50 mm x 20 mm hole in the web
SS3 (U1-U0)	FB0 Span 2	80 mm x 30 mm hole in the center of the web
SS3 (U6-U5)	FB6 Span 1	75 mm diameter hole in the center of the web

3.2 Floorbeams

The inspection of the floorbeams throughout the truss spans of the Old Spences Bridge revealed that although the full length of the bottom flanges exhibit failure of the paint coating and surface corrosion on both the top and bottom faces, they were generally without significant deterioration in the form of section loss, refer to Figure 22. As such, the inspection findings can be focused on the condition of the top flange and the condition of the webs, which have the most significant deterioration.

Figure 22: North Face of Floorbeam 6 in Span 2; Typical Mid-Member Condition

3.2.1 Floorbeam Top Flanges

The deck joints located directly above each floorbeam are presumed to have leaked for many years, resulting in advanced deterioration of the top flange along the full length of the floorbeams, refer to Figure 23. This deterioration includes failure of the paint coating, moderate corrosion product on the underside of the top flange, and formation of pack rust on the top surface of the top flange, refer to Figure 24. As noted in Section 3.1.1.2, the accumulation of pack rust between the top flange and the underside of the concrete deck has resulted in rust jacking, which has lifted the concrete deck off of the stringers. While this jacking has resulted in the redistribution of forces and changes the demand on the concrete deck itself and some deck stringers, the floorbeams are required to support this load in either condition.

Figure 23: South face of Floorbeam 8 in Span 2; Most Extreme Mid-Member Condition (at Deck Joint)

Figure 24: South Face of Floorbeam 8 between DS4 and DS5 in Span 3

In multiple isolated locations, the thickness of the pack rust on the top flange initially appeared to be equivalent to the thickness of the top flange, however, upon further investigation, it was discovered that the observed corrosion was in fact deterioration of a plate located on the top flange of several of the floorbeams. This top plate was not considered during B&T's load evaluation and the corrosion on the plate is not believed to be an immediate concern.

During the 2012 Inspection, locations that were considered to be in a condition worse than 'typical' were cleaned to sound metal over a small area by removing all surrounding corrosion, and the flange thickness was measured to determine the extent of section loss. The measurement of the top flange thickness was made possible as a result of the rust jacking which has lifted the deck and exposed the top face of the top flange. The majority of the thickness measurements taken were between 10 and 12 mm. The most severe locations revealed that about 50% of the original thickness (8 mm) remains in a localized region. Given that the 2012 Updated Load Evaluation Report discussed an allowable loss in thickness of 48% to 52% across the entire width of the top flange it is believed that multiple floorbeams are approaching the permissible limits for section loss. If the bridge is to remain open beyond the end of 2013, it is recommended that these floorbeams be repaired.

Section loss was also identified in localized areas in the top flange of several floorbeams directly over the west top chord of the truss spans. This area is a negative moment region where the top flange of the floorbeam is in tension. Given the reduced demands at this location no repairs are recommended prior to the end of 2013.

3.2.2 Floorbeam Webs

It was noted that due to their increased exposure to the elements, similar to the exterior stringers, the ends of the floorbeams were observed to have significantly more deterioration than the sections under the deck.

Areas of deterioration at the west end of the floorbeams were typically concentrated along the lower 50-100 mm of the web in the cantilever section where it is believed that water has migrated from leaking deck joints. The amount of section loss in these areas was typically 1-3 mm of thickness in the area of the web approximately 450 mm from the end, refer to Figure 25, which is well below the threshold value identified in B&T's evaluation.

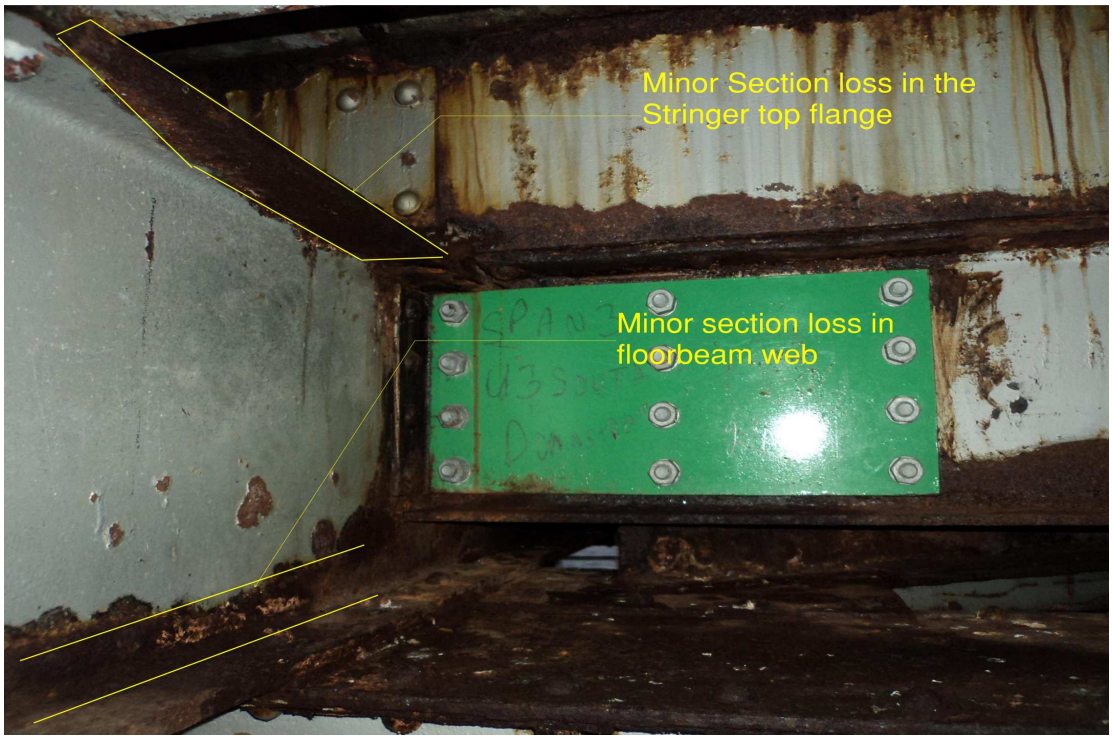


Figure 25: Section Loss in the Flanges of FB2 above the West Truss in Span 3

At the east end of the floorbeams, the webs in the vicinity of the top chord were observed to have varying degrees of section loss and perforations were observed at the eight locations listed in Table 3. These web perforations were identified near the ends of the members either at the overhang portion of the floorbeam or adjacent to the Deck Stringer DS7 connection.

Table 3: Perforations in East End of Floorbeam Web (at Overhang)

Location Number	Span	Floorbeam No.	Approx. Size of Perforation (Long X Tall)
1	Span 1	Floorbeam 4	30 mm x 30 mm
2	Span 3	Floorbeam 2	Several Small Holes
3	Span 3	Floorbeam 5	50 mm x 30 mm
4	Span 3	Floorbeam 6	60 mm x 20 mm
5	Span 3	Floorbeam 8	10 mm x 10 mm
6	Span 4	Floorbeam 4	500 mm x 20 mm
7	Span 4	Floorbeam 8	30 mm x 30 mm
8	Span 4	Floorbeam 10	80 mm x 70 mm

The section loss noted at Location Number 6 in Table 3 is shown below in Figure 26. At this location, the perforation and section loss extend to the top chord of the East Truss where there are significant shear requirements in the floorbeam and it is recommended that the floorbeams be repaired if the bridge is to remain open beyond the end of 2013.

Figure 26: Section Loss in the Web of the East End of FB4 in Span 4

Similar perforations were also observed in the web of two floorbeams above the top chord of the truss on the east side of the bridge. These locations correspond to areas of high shear making the loss of section a greater concern. The floorbeams observed to have web perforations (or substantial section loss) above the top chord are noted in Table 4.

Table 4: Perforations in East End of Floorbeam Web (above Top Chord)

Location Number	Span	Floorbeam No.	Approx. Size Of Perf. (Length X Height)	Notes
1	Span 4	Floorbeam 4	100 mm x 20 mm	a
2	Span 4	Floorbeam 6	150 mm x 15 mm	b
3	Span 4	Floorbeam 8	0 mm	c

Notes:

a) The perforation noted in Table 4 is in addition to the small web perforation at the overhang noted in the Table 3.

b) The perforation noted in Table 4 is in addition to approximately 75% section loss in a 25 mm wide band around the hole.

c) An estimated 50% loss of web thickness in an area 100 mm x 20 mm over the top chord is in addition to the small web perforation at the overhang noted in the Table 3.

The 2012 Updated Load Evaluation Report identified the number of perforations has increased since the 2009 Inspection Report and the observations made during the 2012 Inspection suggest that the load path for bearing in the floorbeam web has been compromised. Recommendations for the timing or repairs are made in the following section.

3.2.3 Floorbeam Assessment Summary

As noted in the previous sections, the extent of the localized section loss in the top flange of several of the floorbeams is believed to be approaching the threshold limits identified in B&T's 2012 evaluation. For the short term, no repairs are recommended for the floorbeam flanges but should be included in budgeting and planning if the bridge is to remain open beyond the end of 2013.

The perforations observed in the webs of some of the floorbeams adjacent to the top chord reduce the shear capacity of the floorbeam and require more timely repair. While a failure of the floorbeam will not result in a collapse of the structure, it represents a safety concern for cars on the bridge and a potential serviceability issue. It is recommended that the floorbeams listed in Table 4 as well as floorbeam 4 in Span 4, refer to Table 3, be a prioritized repair if the bridge is to remain in service beyond the end of 2013.

3.3 Main Truss Members

3.3.1 Top Chords

Although few of the top chords of the truss spans were identified in the 2012 Updated Load Evaluation Report as being members of interest (as shown previously in Figure 5), they were all observed during the detailed inspection to take advantage of the convenience and proximity the scaffolding access provided. The top chords were typically found to be in fair to good condition with some isolated areas in poor condition, due to advanced corrosion and a small number of perforations in the structural steel.

3.3.1.1 Top Chord Members

Eight top chord members were identified in B&T's Updated Load Evaluation Report (U2 to U9 in both Span 3 and Span 4) as to having LLCFs less than 1.5. These areas were reviewed in detail during this inspection and no deficiencies were identified that would require short term repairs. Areas along the top chord that were

identified as being in poor condition were typically located below drain pipes and deck joints, where coating failure and light to moderate surface corrosion was observed. No new significant deterioration or progression of previously identified member section loss was noted. The section loss previously identified at Panel Point U3 in Span 4, and the 80 mm x 60 mm perforation (with surrounding 33% section loss over a 200 mm x 200 mm area) in the top plate of the top chord near floorbeam 6 in Span 2 have not shown signs of significant progression since the 2009 detailed inspection. For the short term life of the bridge no repairs are recommended.

3.3.1.2 Top Chord Batten Plates

The top chords act as compression members and batten plates were used to brace the chord members against buckling by reducing their effective lengths; cover plates were used to add to the cross sectional area that resists the compressive forces. Unlike the chord members themselves, the top chord batten plates and cover plates, located along the top flange of the chords, were found to have widespread coating failure and surface corrosion. At numerous locations, localized areas of minor section loss were identified in these plates, typically adjacent to floorbeams.

Because the locations of section loss are typically adjacent to the floorbeam connections, the localized areas of section loss are not a concern from a stability standpoint as the connection between the floorbeam and the top chord will also serve to provide lateral restraint against local buckling. This is not a concern and short term repairs are not recommended at this time, but the batten plates should be monitored in future inspections.

3.3.2 Bottom Chords

Unlike the top chord, which is partially sheltered by the bridge deck, the bottom chords have greater exposure to the elements. Sections of the bottom chord are also located below deck drains which concentrate the run-off from the bridge deck directly onto portions of the chord members creating an environment even more conducive to corrosion.

3.3.2.1 Bottom Chord Members

As identified during previous inspections, the majority of the bottom chord members are in fair condition, although numerous areas of significant section loss were identified during the 2012 inspection. Significant lengths of several bottom chord members exhibit coating failure with light surface corrosion.

In Span 1, Span 2 and Span 5, areas of section loss were found on the vertical leg of the angles directly below the deck drain locations. These trusses are not as deep as those in Span 3 and Span 4, thus the bottom chord is closer to the drain outlets and are exposed to a higher concentration of drain water as less diffusion takes place in a shorter free-fall. The areas with the most significant section loss as noted in the 2009 detailed inspection are listed in Table 5. The 2012 inspection did not identify any significant increase in number of locations, or severity of section loss at these previously noted locations.

Table 5: Bottom Chord Members with More Advanced Section Loss from 2009 Inspection

Span	East Truss	West Truss
Span 1	L1-L3	L1-L3 L3-L5
Span 2	L3-L5 L5-L7	L5-L7
Span 5	L1-L3 L5-L7	L0-L1 L5-L7

As noted in the 2012 Updated Load Evaluation Report, the bottom chord members listed in Table 5 have spare capacity as demonstrated by the large LLCF numbers and can sustain up to 78% loss of section. While minor section loss was observed in a small number of locations, no areas of section loss approaching the threshold value were observed.

In Spans 3 and 4, localized areas of section loss were frequently observed on the top surface of the top flange and the back side of the channel webs near panel point connections, with those in the vicinity of the deck drains generally in worse condition. Though numerous locations were observed, the degree of section loss at each location is sufficient to consider a decrease in the overall capacity of the members. Several of the bottom chord connections have been re-painted during previous maintenance work which has slowed the corrosion process in many locations.

Furthermore, many of the panel points in Spans 3 and 4 where section loss was identified in the chord member were reinforced with cover plates bolted to the webs circa 2004.

During the 2012 inspection none of the bottom chord members were observed to have any major section loss and no repairs are recommended for the short term life of the bridge. However, if the bridge is to remain open beyond the end of 2013, it is recommended that the deck drains be extended to reduce drainage onto the chord members.

3.3.2.2 Bottom Chord Batten Plates

Many of the batten plates along the bottom chord members were found to have corrosion and section loss, ranging from areas of light surface corrosion to complete section loss. There are also some locations where the extent of loss is not easily quantified due to the corrosion being contained between the member and the batten plate. Because the bottom chord members remain in tension, even under wind load, the batten plates themselves are not crucial to the capacity of the members. While the loss of the batten plates can bring the slenderness of the bottom chord closer to the limitations for tension members, the required extent of loss was not observed for slenderness, or vibrations. The batten plates are not a concern at this time and no repairs are recommended in the short term. However, repairs are recommended if two adjacent batten plates are identified in the future with extensive section loss.

3.3.2.3 Bottom Chord Panel Points

A corrosion pattern was identified at the even numbered bottom chord panel points in Spans 3 and 4. At these locations, the gusset plates connecting the vertical and diagonal members to the bottom chord were observed to have areas of section loss immediately above the level of the bottom chord top flange and in some locations extending up behind the Truss Diagonal, as seen in Figure 27. It is believed that the section loss was originally caused by the accumulation of moisture and debris in these locations, and perhaps the corrosion was not completely removed when repainted resulting in the continuation of deterioration and the corrosion staining observed today. In the majority of locations, the section loss was only noted on the interior gusset plate although areas of section loss were also noted in a small number of exterior gusset plates.

Figure 27: Typical Section Loss Observed along Bottom Chord Gusset Plates showing Extent to Section Loss above Level of the Top Flange and the Section Loss Extending up the Diagonal

As illustrated in Figure 4, the 2012 Updated Load Evaluation Report noted a concern with the capacity of the uncorroded gusset plates at the following locations:

- Panel Points L0, L4, L6 and L10 in Span 3; and
- Panel Points L0, L4, L6 and L10 in Span 4.

The gusset plates identified above also follow the typical corrosion pattern with approximately 3 mm of section loss in the plate at the top chord and behind the truss diagonal. While corrosion was observed in the bottom chord panel points, the level of section loss was not sufficient enough to recommend repairs prior to the end of 2013 but should be included in budgeting and planning if the bridge is to remain open beyond the end of 2013.

3.3.3 Verticals

The vertical members in the truss spans were found to be generally in good condition with a small number of isolated areas of coating failure and surface corrosion observed.

Figure 28: General View of Vertical Members U0-L0 in Span 3

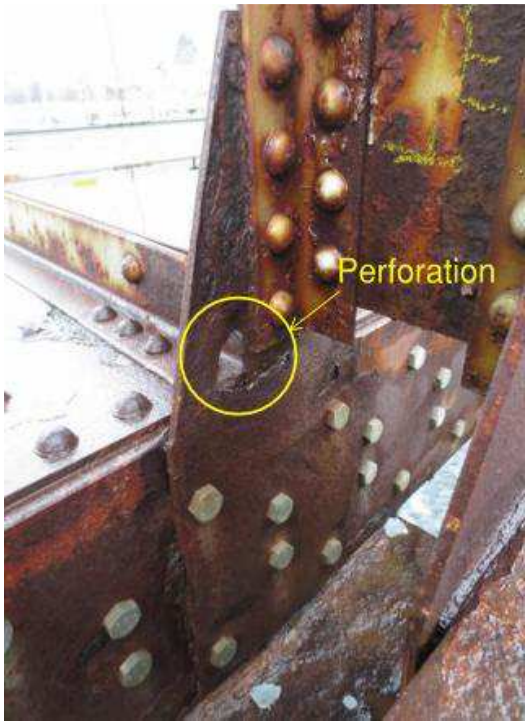


Figure 29: Perforation in Gusset Plate of L1E in Span 4

The connections between the verticals and the top chord were typically observed to be in good condition while the connections to the bottom chords were found to be in fair to poor condition. Specifically, the gusset plates connecting the vertical members to the bottom chord in the vicinity of the deck drains were found to be in poor to very poor condition with multiple perforations identified. In several locations pack rust was observed in the joint between the gusset plate and the back side of the bottom chord channel webs.

Although some of the vertical member gusset plates have significant perforations and/or exhibit pack rust, there is no immediate structural concern. These gusset plates connect zero force members to the bottom chord of the truss and, while they provide some stability to the bottom chord, they are considered primarily for serviceability requirements (i.e. they carry very little load).

Vertical U0-L0 in Span 3 was identified in the 2012 Updated Load Evaluation Report, as having a LLCF less than 1.5, due to wind loading, when no corrosion was considered. This member, shown in Figure 28, carries the vertical load from half of both Span 2 and Span 3 to the bearings at Pier 2. No deterioration was observed in

vertical U0-L0 during the 2012 Inspection and no repairs are recommended to the member at this time. However, if the bridge is to remain open beyond the end of 2013, it is recommended that the members be strengthened.

3.3.4 Main Diagonals

Several of the truss diagonals were highlighted in the 2012 Updated Load Evaluation Report as having LLCF's between 1.0 and 1.5 however the detailed inspection did not identify any deficiencies in these members. The diagonal members were found to be generally in good condition with only limited areas of coating failure and light surface corrosion observed and no repairs are recommended at this time.

3.3.5 Main Truss Members Assessment Summary

The main truss members are in fair condition with the most severe deterioration along the bottom chord panel points and no areas of section loss exceeding the tolerable limits identified in the Load Evaluation Report were observed. Therefore, no recommendations for immediate repairs are recommended for the main truss members. However, it is recommended that the deck drains be extended if the bridge is to remain open beyond the end of 2013.

3.4 Lateral Load Carrying Members

The lateral bracing members were typically found to be in fair condition with areas of coating failure and surface corrosion observed on multiple members. While relatively few issues were found with the members themselves, the gusset plates that connect the lateral bracing to the chord members were typically found to be in fair to poor condition.

3.4.1 Bottom Chord Lateral Bracing

The 2012 Updated Load Evaluation Report identified 24 bottom chord lateral bracing members with a LLCF below 1.5; four of which were below 1.0 and form the focus of the 2012 inspection.

In general the members were observed to have minor coating loss with isolated areas of minor surface corrosion. However, the lateral bracing gusset plates were observed to have numerous areas of section loss and multiple perforations. These gusset plates are susceptible to debris build up that can retain moisture on top of the plates, leading to section loss and a reduction in the capacity of the lateral system. The condition and recommendation of several members identified in the 2012 Updated Load Evaluation Report are listed in Table 6:

Table 6: Bottom Chord Lateral Bracing Condition

Span	Member	Condition	Recommendation
3	Strut at L1	The strut has some surface corrosion and slight section loss on the west gusset plate.	No repairs recommended
3	Strut at L2	The strut has been previously replaced. The gusset plate has minimal section loss in most of the surface but there is a 20 mm diameter hole in the plate outside of the load path.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
3	Strut at L3	The strut has surface corrosion. The gusset plate has a 4 mm section loss in the load path and a 60 mm diameter hole not outside of the load path.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
3	Lateral Bracing between L8 and L9	West gusset plate at L8 has corrosion with light section loss between the bracing and the bottom chord.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
3	Strut At L8	The strut has been previously replaced.	No repairs recommended
3	Lateral Bracing between L9 and L10	West L10 gusset plate has minor section loss in the load path.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
4	Strut at L1	The strut has some surface corrosion. The west gusset plate has surface corrosion with minor section loss.	No repairs recommended
4	Strut at L2	The strut has been previously replaced.	No repairs recommended
4	Strut at L7	The strut has some surface corrosion and the west gusset plate has surface corrosion with minor section loss.	No repairs recommended
4	Lateral Bracing between L8 and L9	The west L9 gusset plate has surface corrosion with minimal section loss except in the area near the strut.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
4	Strut at L9	The strut has some surface corrosion. The gusset plate connection is starting to develop pack rust between the gusset plate and the strut with some pitting in the gusset plate, refer to Figure 30.	Recommend gusset plate be repaired if bridge to remain open beyond end of 2013.
4	Lateral Bracing between L9 and L10	The bracing has some isolated areas with surface corrosion.	No repairs recommended

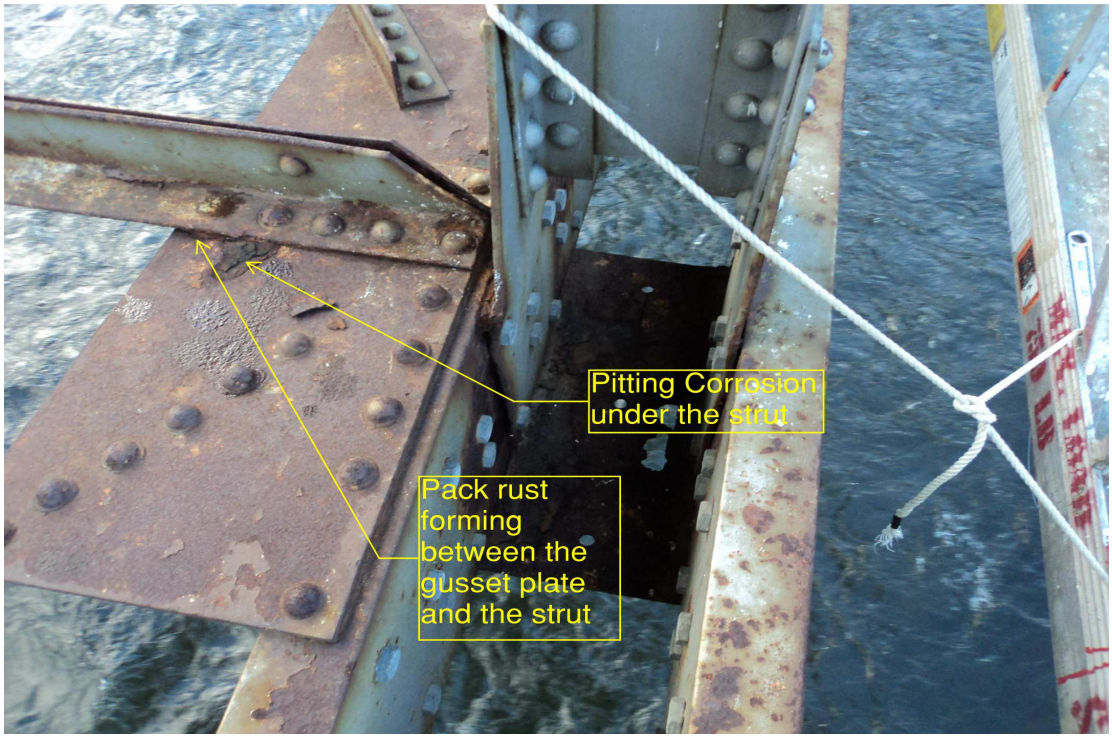


Figure 30: L9 Lateral Strut West Gusset Plate

3.4.2 Top Chord Lateral Bracing

The 2012 Updated Load Evaluation Report identified twelve top chord lateral bracing members that had a LLCF below 1.5. During the 2012 inspection, the gusset plates were typically observed to have complete coating failure on their surfaces with light surface corrosion and minor section loss. The lateral bracing members were observed to be in better condition than the gusset plates with most of the coating in place and small areas of surface corrosion. No recommendations for repairs to either the members or the gusset plates are made at this time.

3.4.3 Sway Bracing

The general condition of the sway bracing members and gusset plates is fair, however many of the sway bracing members located directly below the deck joints were observed to be in poor condition. Coating failure and surface corrosion were widespread on both the members and gusset plates and numerous perforations were identified in both members and gusset plates, refer to Figure 31 and Figure 32. A list of locations where perforations were observed in the sway bracing members is included in Table 7.

Figure 31: Perforations in the Mid Height Strut Top Flange at PP0 in Span 4

Figure 32: Lower Sway Bracing Gusset Plate at L6W, Span 4, with Four 20 mm Diameter Holes

Table 7: Sway Brace Members with Major Corrosion and Perforations

Span	Member	Condition	Recommendation
3	PP4 - Mid-height transverse strut	25 mm diameter perforation in the horizontal leg.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP6 - Mid-height transverse strut	2-25 mm x 30 mm perforations in the horizontal leg.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP8 - Mid-height transverse strut	Section loss for roughly 20% of member thickness near gusset plate.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP10 - Mid-height transverse strut	30 mm diameter perforation in the horizontal leg.	Recommended to be repaired if bridge to remain open beyond end of 2013.

4	PP0 - Mid-height transverse strut	20 mm x 150 mm and 20 mm diameter perforation in the horizontal leg.	Recommended to be repaired if bridge to remain open beyond end of 2013.
4	PP6 - Mid-height transverse strut	4-30 mm diameter perforations in the horizontal leg.	Recommended to be repaired if bridge to remain open beyond end of 2013.

Sway bracing gusset plates with observed section loss or perforations are listed in Table 8. The majority of these perforations were located outside the load path and no repairs are recommended prior to the end of 2013 but should be included in planning and budgeting if the bridge is to remain in service beyond the end of 2013. The gusset plate at the base of the sway bracing at PP6 in Span 4 contains perforations directly in the load path and recommendations are made in Section 3.4.4 for its replacement.

Table 8: Sway Bracing Gusset Plates with Observed Section Loss and Perforations

Span	Member	Condition	Recommendation
3	PP0W - Base	50 mm perforation below top sway brace.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP4E - Base	Roughly 50% section loss of plate thickness partially in the load path.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP8E - Mid Height	Roughly 20% section loss of plate thickness in the load path.	Recommended to be repaired if bridge to remain open beyond end of 2013.
3	PP10E - Base	10 mm DIA perforation in the load path.	Recommended to be repaired if bridge to remain open beyond end of 2013.
4	PP0W - Mid Height	4 mm section loss above the top sway brace.	Recommended to be repaired if bridge to remain open beyond end of 2013.

4	PP4E - Base	Two 5 mm pinhole perforations in the load path.	Recommended to be repaired if bridge to remain open beyond end of 2013.
4	PP6W - Base	Four 20 mm diameter perforations.	Recommended to be repaired if bridge to remain open beyond end of 2013.
4	PP6E - Base	Roughly 20% section loss of thickness in load path.	Recommended to be repaired if bridge to remain open beyond end of 2013.
5	PP0W-Mid Height	Two 40 mm x 130 mm perforations.	Recommended to be repaired if bridge to remain open beyond end of 2013.

3.4.4 Lateral Load Carrying Members Assessment Summary

The 2012 Updated Load Evaluation Report listed many lateral bracing members that had a LLCF that were less than 1.5 in an un-corroded condition due primarily to changes in wind loading between the code in place at the time of original construction and the current code. These members were inspected and the condition of the members ranges from fair to good; no repairs are recommended at this time.

The West gusset plate for the lateral strut at PP9 in Span 4 was identified as having a LLCF less than 1.0 in an un-corroded state. During the 2012 inspection, a localized area of section loss was identified on the gusset adjacent to the strut connection which may further reduce the capacity of the connection. The section loss will reduce the capacity of the member. It is recommended that the repair of the gusset plate be prioritized if the bridge is to remain open beyond the end of 2013.

Due to perforations in the load path, it is recommended that the lower sway bracing gusset plate at PP6W, Span 4, be replaced at the same time as the PP9W if the bridge is to remain in service beyond the end of 2013.

3.5 Bearings

Two types of bearings have been used on the Old Spences Bridge. At Piers 1 and 5 the superstructure is supported by sliding bearings. At Pier 3 the bridge truss is supported by roller bearings while Piers 2 and 4 have fixed bearings.

3.5.1 Sliding Bearings

For the three shorter truss spans (and the girder spans) the sliding bearings consist of two steel plates sliding across one another. One of the steel plates is outfitted with a steel tab while the other plate has a groove machined into it. This tab prevents transverse displacement of the plates while allowing longitudinal movement. As identified in previous inspections, pack rust was observed in the gap between the two plates. It is believed that this pack rust severely limits the amount of movement that can be accommodated by the bearing and it is likely that the bearings no longer perform as originally intended. No repairs are recommended for the bearings prior to the end of 2013 however, if the bridge is to remain in service beyond this time, it is recommended that these areas be cleaned and recoated.

3.5.2 Roller Bearings

The roller bearings for the longer spans are located at Pier 3 and consist of a pin assembly located on top of a nest of five steel rollers, refer to Figure 33. Three anchor bolts, situated in slotted holes, connect the pin assembly to the bearing base plate and prevent uplift. The ends of the rollers are visible through holes in guide plates on either side of the bearings, although one of the rollers on the east side of the southwest bearing for Span 4 was found to have come out of the hole in the guide plate.

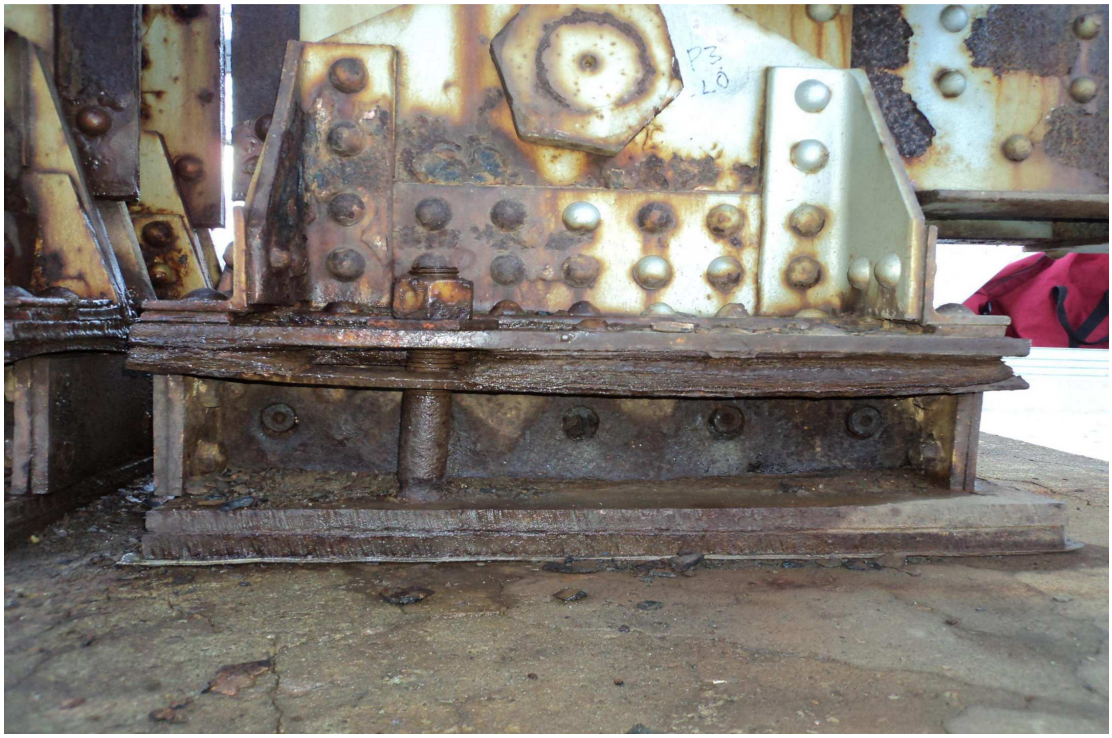


Figure 33: East Face of the West Bearing on Pier 3 for Span 4

All four of the roller bearing bases at Pier 3 (two for Span 3 and two for Span 4) were observed to be widely covered in surface corrosion and pack rust was identified between many of the plates in the bearing assemblies. No obvious signs of longitudinal movement were observed and there is a possibility that the bearings have seized, a theory supported by the undisturbed debris accumulations observed around the bearings. The washer beneath the northwest anchor bolt on the southwest bearing of Span 3 was found to be tight against the surface of the pin assembly bearing plate and this condition may restrict the ability of the bearing to move. The cracked top surface of Pier 3 may also suggest that the longitudinal movement of the bridge truss is being transferred into the pier.

Wear patterns on the pin assembly and rotated keeper nuts on the outside of the bearings suggest that the bearings still allow rotation about the transverse axis, refer to Figure 34.

Many of the anchor bolts were found to be out of plumb (i.e. they have been bent away from vertical), refer to the dashed lines in Figure 34. At a given bearing, the anchor bolts did not appear to be bent in the same directions, which implies that misalignment during installation and forces transferred to the anchor bolt through seized bearings are likely what has resulted in the anchor bolts being out of plumb. It is important to note that the top of the anchor bolt on the west side of the southeast bearing of Span 4 has sheared off and was found lying on the top of the pier cap. Additionally, an area of reduced cross section was observed in the southwest anchor bolt at the northwest bearing of Span 3 and in the east anchor bolts at the southeast bearings of Span 4.

For the short term, no repairs are recommended for the roller bearings at pier 3. However, if repairs are made to the concrete bridge piers it is recommended that the roller bearings be rehabilitated.

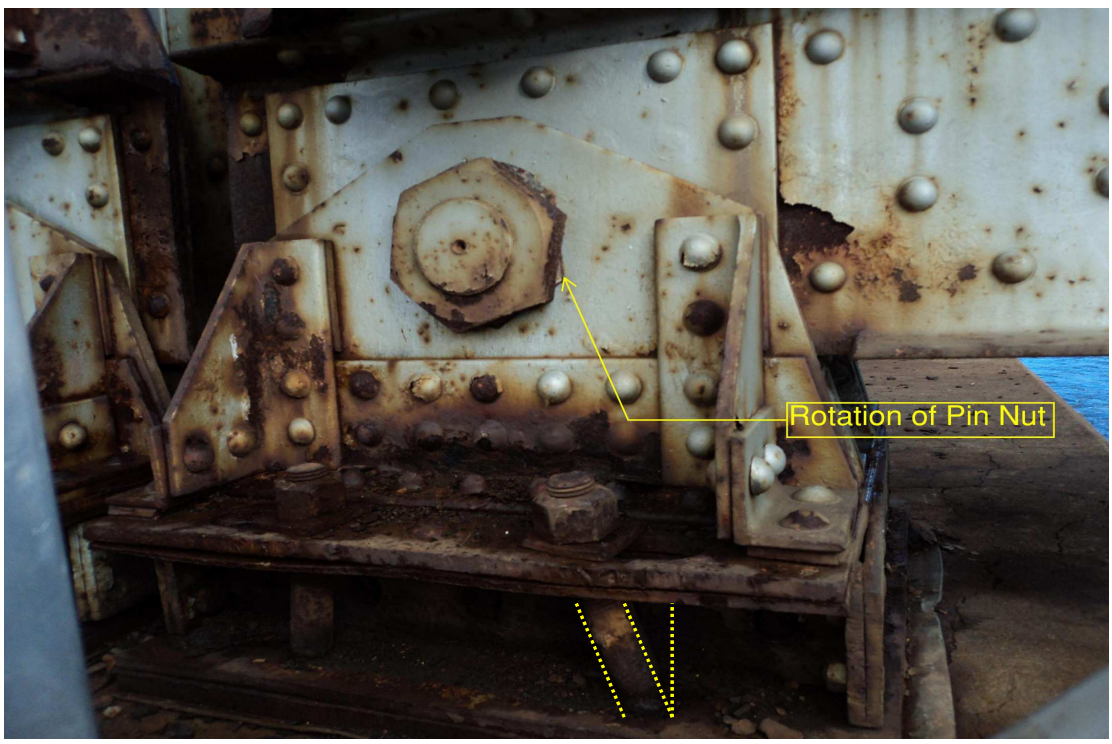


Figure 34: West Face of the West Bearing on Pier 3 for Span 3 - Rotation of the Nut Relative to the Truss Suggests Rotation

3.5.3 Pier 2 Bearing

The 2012 Updated Load Evaluation Report identified that there was an uncertainty regarding the number of rivets installed. The number of rivets installed in the outer doubler plate of the vertical truss member is less than what is shown in the original shop drawings. The vertical doubler plates at Pier 2 have rivet holes that have been filled with weld material. The holes appear to have been filled because there is interference with the holes in the outer doubler plate of the vertical truss member and the angle for the interior diaphragm, see Figure 35. The holes appear to be filled with weld rather than a headless rivet because there are filled holes that are in interior plate that do not coincide with the outer plate. Therefore, it is recommended that the shear capacity of the connection should be calculated using the visible number of rivets.

3.5.4 Bearings Assessment Summary

Although it appears that the bearings are seized from the 2012 Inspection, it is recommended that no repairs are required for short term use of the bridge as the flexibility of the substructure appears to be sufficient to accommodate the thermal movements of the structure. However, if repairs are made to the concrete piers, as discussed in Section 3.6, it is recommended that the roller bearings be rehabilitated to prevent the longitudinal forces being transferred into the superstructure.

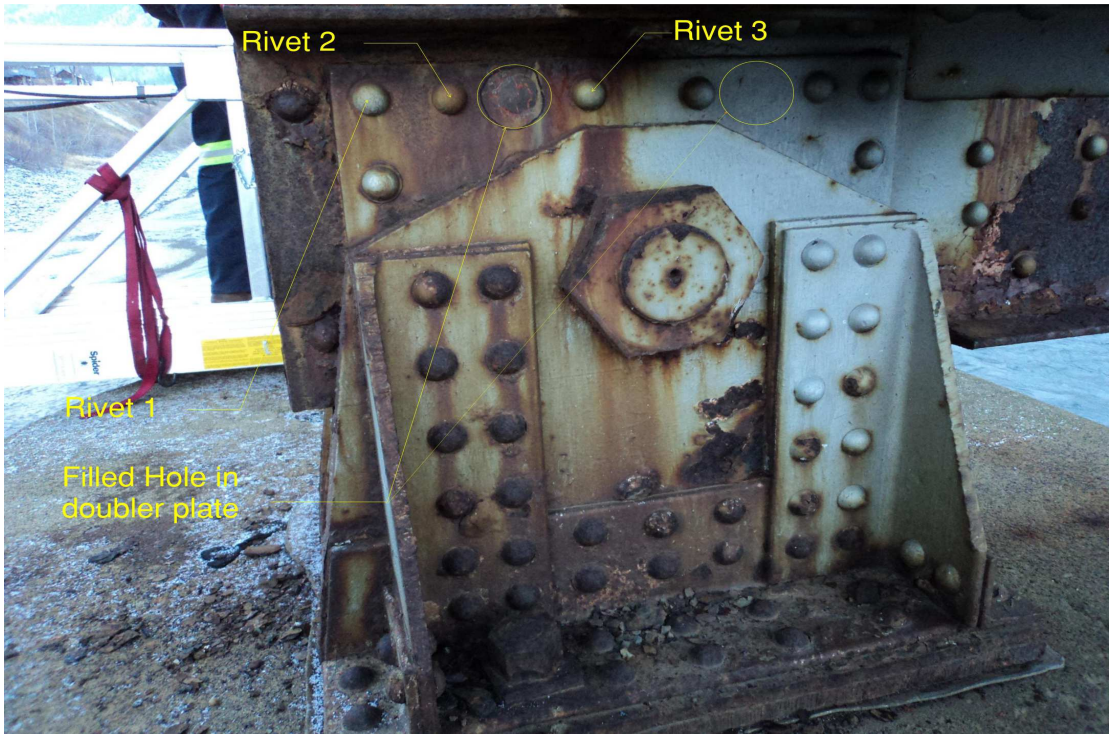


Figure 35: East Face of the West Bearing at Pier 2 for Span 3

3.6 Concrete Piers

The six concrete piers that support the bridge were inspected using a combination of visual assessment and hammer sounding from the ground and from swing staging.

The condition of the Piers 1 through 6 is mostly unchanged from the inspection that was performed by BCMOT in 2011. It was observed that apparent construction joints or horizontal cracks are present in the concrete piers and may limit the ability of the unreinforced concrete piers to resist longitudinal bending moments due to loads such as wind and seismic.

No repairs to the concrete piers are recommended for the short term serviceability of the bridge. However, if the bridge is to remain open beyond the end of 2013, it is recommended that repairs be carried out on the piers to increase their strength.

4 Closing

B&T's scope of work for the Old Spence's Bridge was to perform a detailed inspection of the steel truss spans and concrete piers of the structure to confirm several assumptions made in the 2012 Updated Load Evaluation Report with the aim to provide a level of confidence to BCMOT for keeping the bridge open until 2013 December 31.

Buckland & Taylor's 2012 detailed inspection of the Old Spences Bridge reaffirmed that overall the bridge is in fair condition, and identified many areas that are in poor condition. The majority of the areas of concern, as identified in the 2012 Updated Load Evaluation Report, did have some small amounts of section loss but were within the acceptable tolerances based on calculated capacities.

While the focus of the 2012 Inspection was to identify components of the bridge requiring repair in order to keep the structure open until the end of 2013, it should be noted that numerous repairs will be required to many areas of the structure if the bridge is to remain open beyond the end of 2013 in order to ensure a serviceable structure. These include:

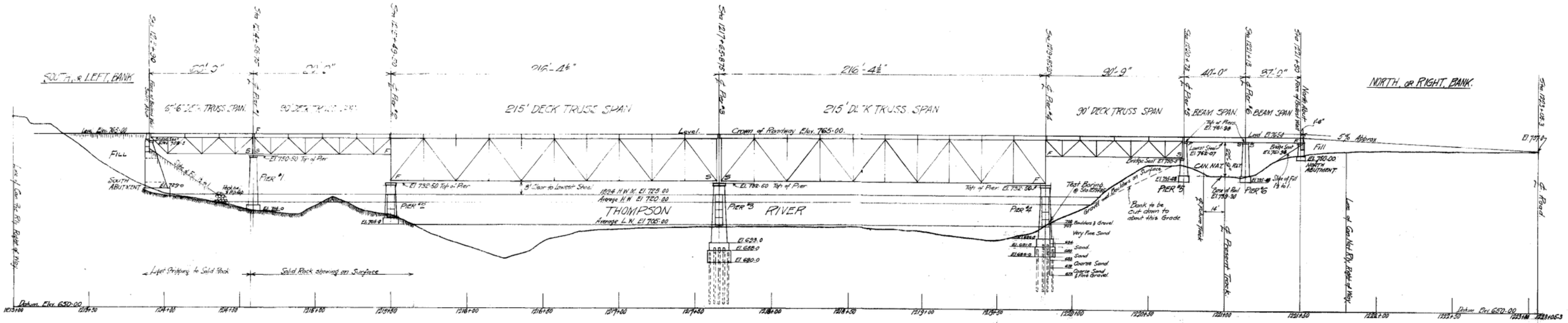
- Repair deteriorated deck stringers;
- Repair deteriorated sidewalk stringers;
- Repair floorbeam flanges;
- Repair floorbeam webs;
- Replace sway bracing gusset plates;
- Repair the concrete piers;
- Extend deck drains;
- Replace bottom chords batten plates on an as needed basis; and,
- Replace main truss gusset plates.

Of the items listed above, four items are considered as having the highest priority and are listed in Table 9.

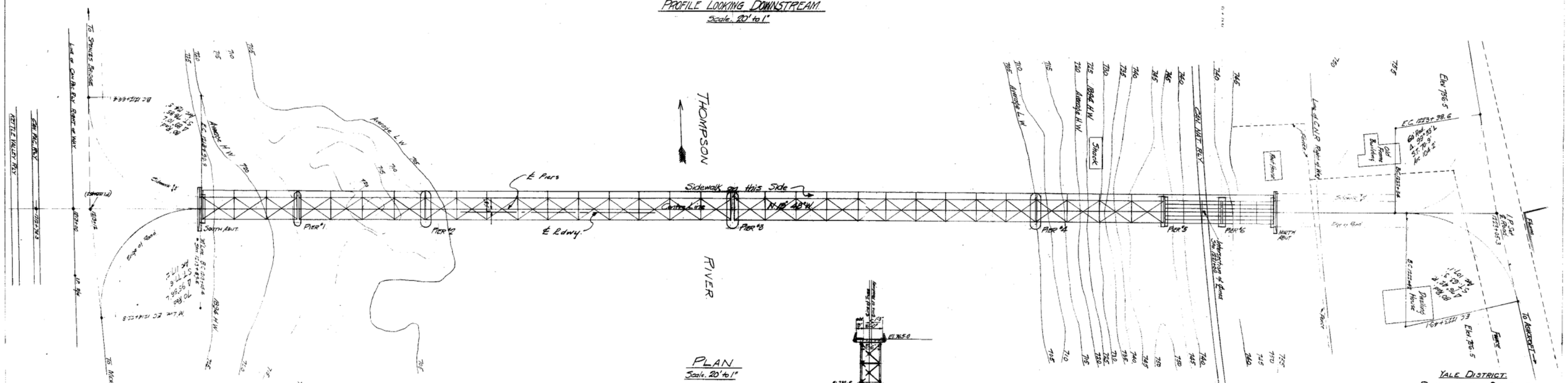
Table 9: Prioritized Repairs

Span	Member	Repair	Reason
S4	Floorbeam 4	Web above the East Truss.	The size of the perforations in the web is increasing and the load path through the floorbeam is becoming compromised (high shear location).
S4	Floorbeam 6		
S4	Floorbeam 8		
S4	L6 West sway brace gusset plate	Replace gusset plate and remove and replace rivets with A325 bolts.	The perforations extend almost the full width of the gusset plate and are compromising the load path.

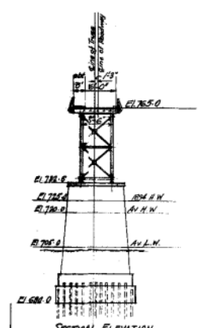
Appendix A
Old Spences Bridge General Arrangement



PROFILE LOOKING DOWNSTREAM
Scale. 20' to 1"



PLAN
Scale. 20' to 1"



SECTIONAL ELEVATION AT PIER #3

INDEX TO DRAWINGS

Sheet #1	Lay-out Plan
" #2	Overhead Crossing C.N.R.
" #3	Details of Piers #2, 3 & 4
" #4	Details of North Abutment
" #5	Details of South Abutment
" #6	Details of Miscellaneous Steel.

Formerly Dwg. No. 757-1

YALE DISTRICT.

PROPOSED BRIDGE AT SPENCES BRIDGE.

— OVER THE THOMPSON RIVER. —

LAY-OUT PLAN.

Scale-as noted

GOVERNMENT OF BRITISH COLUMBIA.
DEPARTMENT OF PUBLIC WORKS.
VICTORIA, B. C.

Drawing No. 2471 Sheet No. 1
Made by: C.D.M.C. Date: May 1928
Traced by: Date:
Checked by: J.T.H. Date: Oct. 3, 1929
Examined and Recommended for Approval: E. Davison
APPROVED: S. Mathers
Deputy Minister and Public Works Engineer.
Assistant Public Works Engr.