BC Ministry of Transportation & Infrastructure

Old Spences Bridge No. 2411

Inspection Report

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Executive Summary

The Old Spences Bridge was constructed in 1931 and crosses the Thompson River providing a link between Highway 8 and Highway 1 in the Community of Spences Bridge, BC. In 1962, a new bridge was constructed approximately 900 m downstream that also connects Highway 8 and Highway 1.

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 a single span of 21.0 m (69 feet), two spans of 27.7 m (91 feet) and two spans of 65.8 m (216 feet). The girder spans are 11.3 m (37 feet) and 12.2 m (40 feet) making the total length of the bridge 231.6 m (760 feet). Six concrete piers and two concrete abutments support the bridge.

Annual inspections of the Old Spences Bridge have been performed for many years and following the 2002 inspection the bridge was posted with a 25 tonne load limit. During the 2008 inspection, significant deterioration, corrosion and holes were identified in heavier structural components. Based on the 2008 visual inspection the bridge was closed to all vehicular traffic in 2009 in order to ensure public safety.

Subsequent to closing the crossing, the British Columbia Ministry of Transportation and Infrastructure (BC MoT) retained Buckland & Taylor Ltd. (B&T) to carry out a detailed inspection and load capacity evaluation of the structure. As part of their 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.

This report contains observations made during B&T's 2009 inspection and makes recommendations regarding areas to focus on as part of the bridge for evaluation, as well as listing items for maintenance and future inspection. Recommendations for rehabilitation items have also been provided based on the inspection findings. This report does not address the cost effectiveness of carrying out the items identified above.

B&T Report No. 1884-RPT-SPE-002-0, entitled "Load Capacity Evaluation & Rehabilitation Options - Old Spences Bridge No. 2411" summarizes the findings of the load evaluation of the bridge, makes recommendations regarding conceptual rehabilitation options, and summarizes cost estimates to restore the bridge to a range of acceptable levels of reliability.

B&T's 2009 Inspection of the Old Spences Bridge found that overall the bridge is in poor condition, but also identified many areas that are in very poor condition. Some of the areas in very poor condition may affect the capacity of the bridge to safely carry vehicular, pedestrian, or snow loads. Since it is not possible to establish the load carrying capacity of the bridge based

on a visual inspection, a load capacity evaluation of the bridge must be carried out to determine whether it is safe to reopen the bridge to traffic and what level of traffic (i.e., load posting) can safely use the bridge.

The most significant findings and recommendations based on this inspection are as follows:

- Widespread coating failure was observed on the bridge steel. Trans Canada Coating Consultants Ltd. were retained to inspect and to provide an estimate of remaining service life of the current coating. Based on the results of the inspection, it has been determined that in order “to gain useful life for the bridge the corrosion must be slowed or stopped”. Cost estimates for two recoating options are included in this report.
- Localized areas of section loss and perforations were observed in multiple stringers, floorbeams and bracing members. Evaluation criteria have been included in this report as a guide for determining the capacity of the components and rehabilitation items for these members.
- Pack rust and rust jacking were found to have changed the support conditions of the concrete deck. It has been recommended that rehabilitation options be developed for the concrete deck.

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1 Introduction

The Old Spences Bridge was constructed in 1931 and crosses the Thompson River providing a link between Highway 8 and Highway 1 in the Community of Spences Bridge, BC. In 1962, a new bridge was constructed approximately 900 m downstream that also connects Highway 8 and Highway 1.

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 a single span of 21.0 m (69 feet), two spans of 27.7 m (91 feet) and two spans of 65.8 m (216 feet). The girder spans are 11.3 m (37 feet) and 12.2 m (40 feet) making the total length of the bridge 231.6 m (760 feet). Six concrete piers and two concrete abutments support the bridge. For reference, a General Arrangement drawing of the bridge is included in Appendix A.

Annual inspections have been performed for many years and in 2002, the inspection identified corrosion damage in structural members resulting in the bridge being posted with a 25 tonne load limit. During the 2008 visual inspection, significant deterioration, corrosion and holes were identified in heavier structural components. Based on this visual inspection the bridge was closed to all vehicular traffic in 2009 in order to ensure public safety.

1.1 Inspection Scope

Subsequent to closing the crossing to vehicle traffic, BC MoT retained Buckland & Taylor Ltd. (B&T) to carry out a detailed inspection and 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. This report summarizes the findings of the detailed inspection of the Old Spences Bridge. Recommendations have been included in this report and have been classified as:

- Evaluation Items, items included in the evaluation portion of B&T's scope;
- Rehabilitation Items, items included as conceptual rehabilitation options;
- Maintenance Items, items expected to be included in annual maintenance. These items are only applicable should BC MoT choose to re-open the bridge; and
- Inspection Items, items that require continued monitoring.

1.2 Inspection Procedure

Due to the fact that the bridge is closed to vehicle traffic, it was not possible to inspect the bridge using an under-bridge inspection vehicle. For inspection of the below deck portions of the bridge, as well as the sides of Piers 1, 3 and 5, swing stages and bridging units were used, refer to Figure 1 and Figure 2. 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 3.

Figure 1: Swing Stage Access

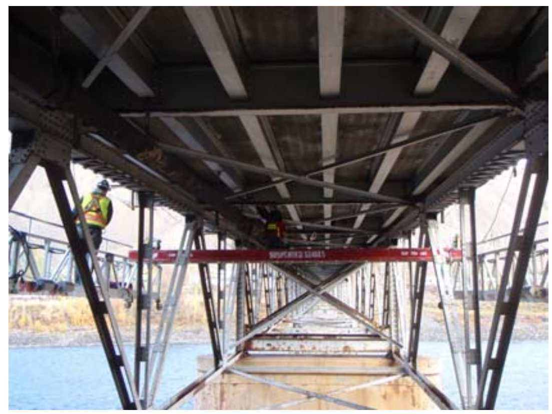


Figure 2: Bridging Unit between Swing Stages

The bridge deck and sidewalk were chain-dragged in an effort to identify delaminations in the concrete and the railings along both sides of the deck were visually inspected from the bridge deck. The North and South Abutments, Piers 1, 2, 4, 5 and 6, and portions of Spans 1 and 7 were accessed from the ground.

Previous inspections had raised concerns with the degree of corrosion and deterioration of various bridge members. As a result, it was of particular interest to assess areas of significant deterioration during the inspection. In order to accomplish this, B&T utilized General Electric DMS2 ultrasonic thickness gauges to measure the thickness of sound material. In areas where the surface condition would not permit the use of the thickness gauge, an electric hand grinder was used to prepare an area on the surface to receive the thickness gauge.

Figure 3: Scaffolding System Used to Support Swing Stages

2 Bridge 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. For ease of reference in this report, the bridge components have been labeled and a drawing showing the numbering convention is included in Appendix B.

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 4 to Figure 7, and are described in more detail in the subsections that follow.

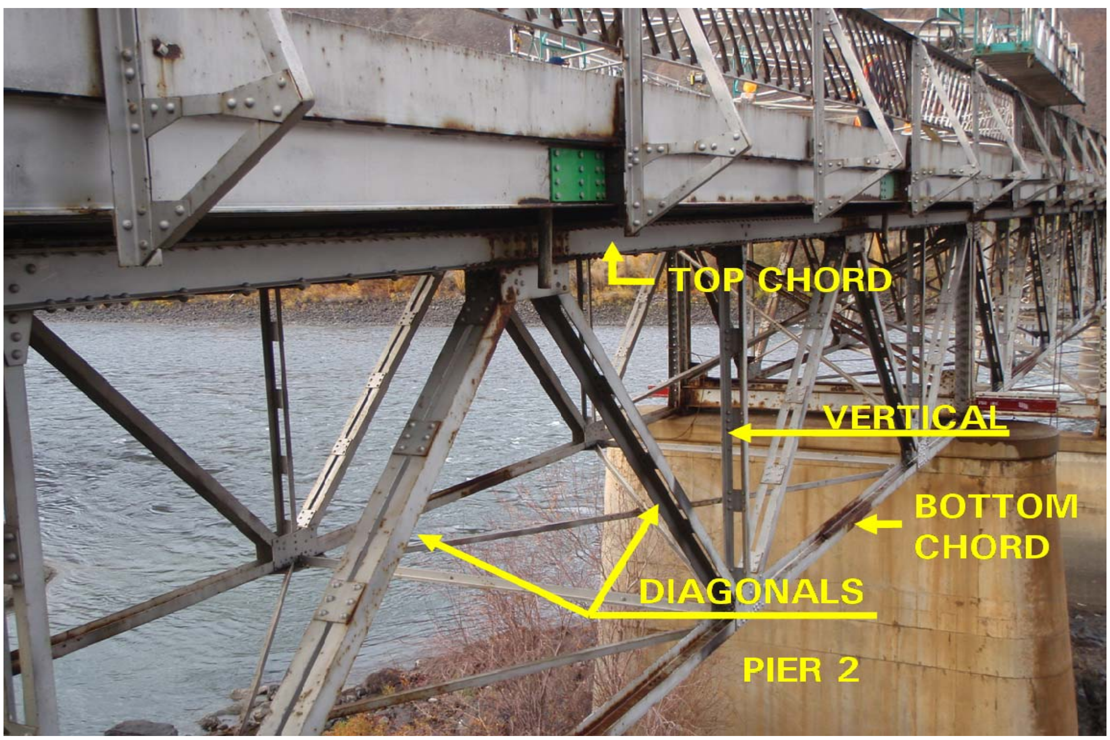


Figure 4: View of Typical Truss Span Showing Vertical Load Carrying Members

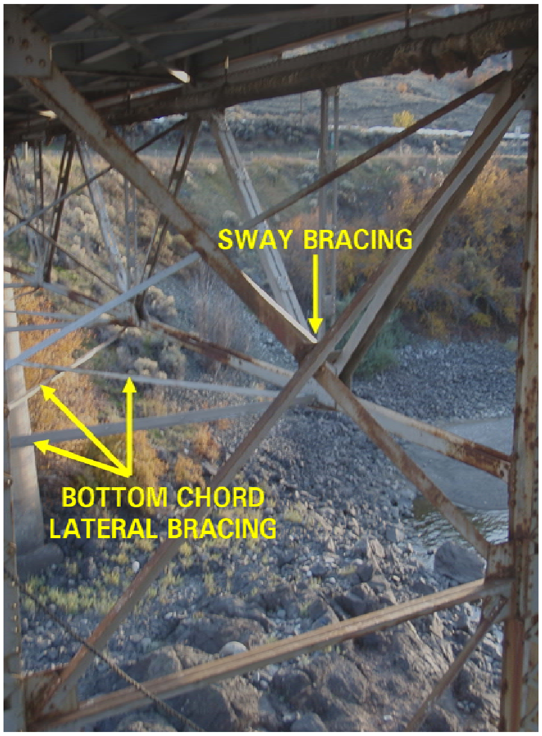


Figure 5: View of Typical Truss Span Showing Lateral Load Carrying Members

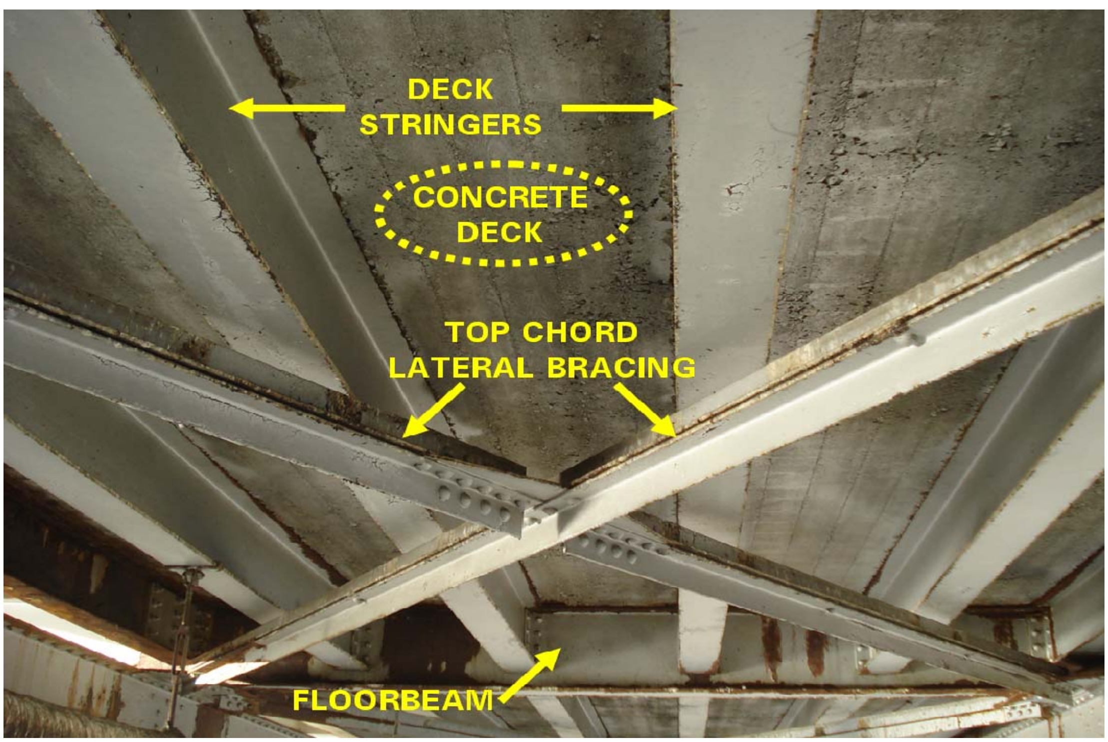


Figure 6: View of Typical Floor System in Truss Span

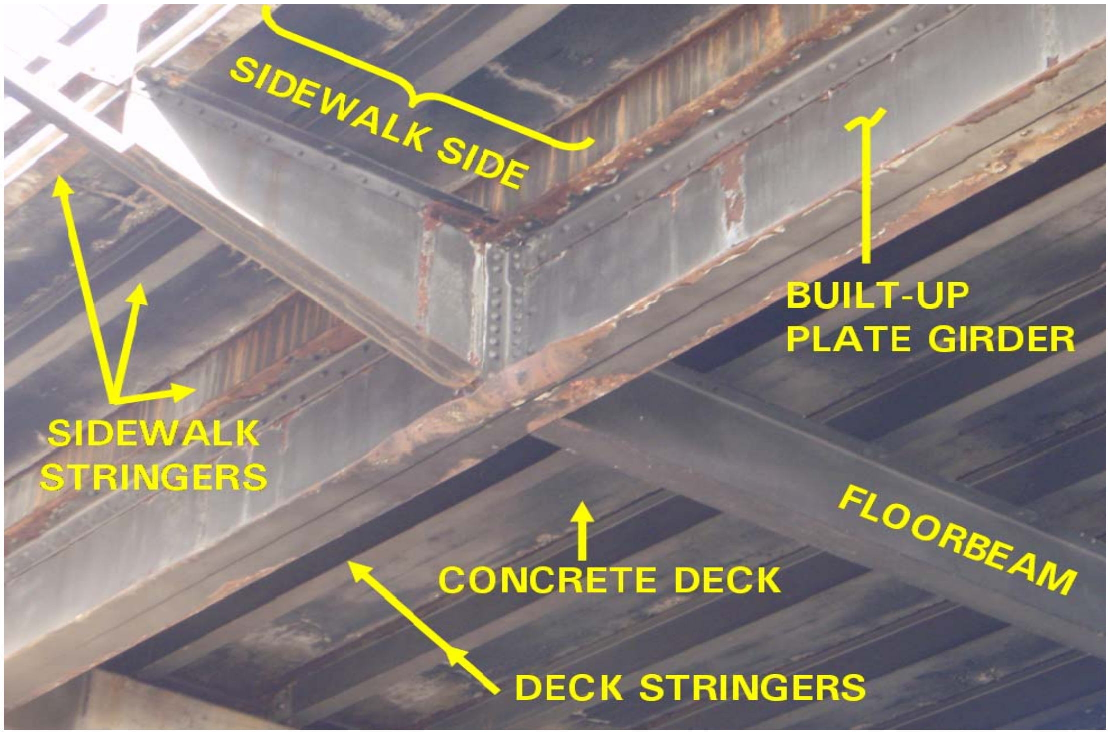


Figure 7: View of Typical Floor System in Girder Spans

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 differ 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 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 formed with single angles as cross-bracing members. 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). 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.

There is a 1220 mm (4 foot) 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.

For the purpose of this report, the stringers have been designated as either deck stringers (DS) or sidewalk stringers (SS) and have been numbered from west to east, refer to Figure 8 and Figure 9.

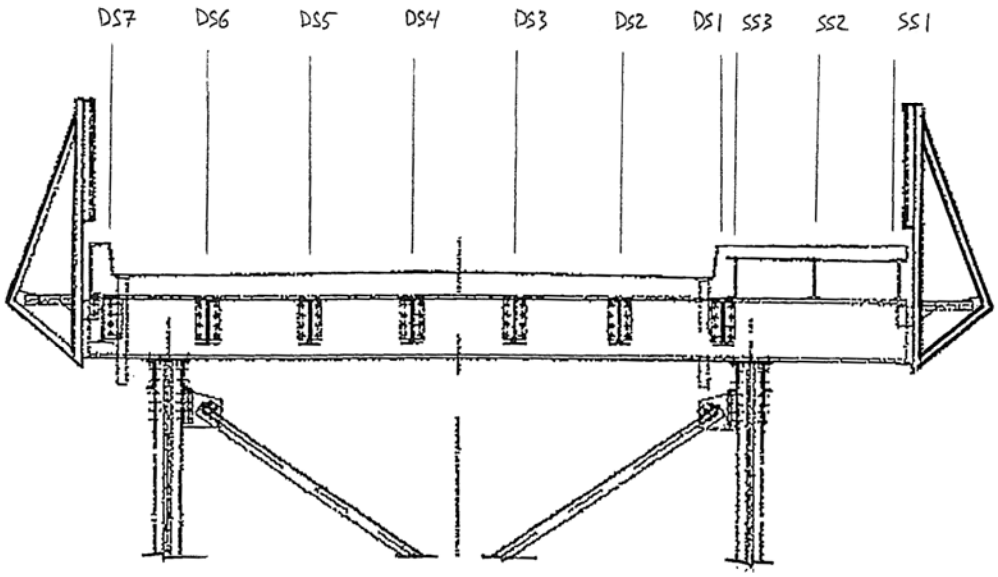


Figure 8: Stringer Arrangement in Truss Spans

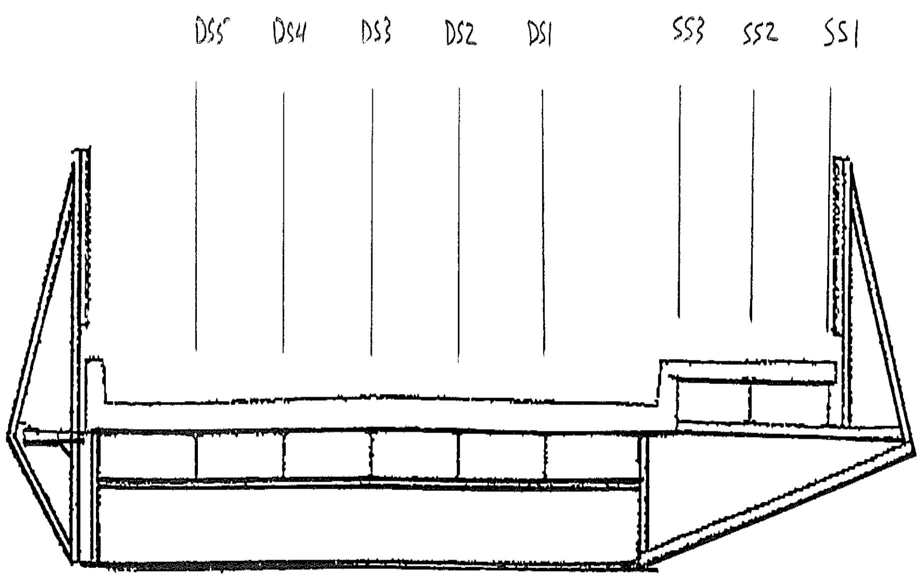


Figure 9: Stringer Arrangement in Girder Spans

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 and 11.3 m (40 and 37 feet), 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.

3 Inspection Findings

The findings of this inspection are presented in the following sections of this report and have also been summarized on the standard BC MoT Bridge Management Information System (BMIS) Condition Inspection Sheets included in Appendix C**.**

3.1 Approach Roadways and Embankments

The approaches to the Old Spences Bridge are constructed on fill. The North Approach has a 5% slope while the South Approach has a 0% slope. At the time of the inspection, a Maintenance Contractor was completing the installation of gates.

It was observed that the soil on the east and west sides of the North Approach immediately behind the abutment was sloughing away. On the east side, the sloughing soil has undermined the roadway and the guardrail post. A tape measure was used to determine the extent of the undermined area and it was found that a void extended approximately 785 mm (31”) under the roadway from the east side, refer to Figure 10. While a void was not identified on the west side, the ground was observed to have sloughed significantly resulting in a vertical face along the west edge, refer to Figure 11. It is recommended that the sides of the North Approach roadway immediately behind the abutment be reinforced and that the void under the roadway be filled (Maintenance Item M-1).

Figure 10: Void and Undermined Post at North Approach-East Side

Figure 11: Sloughing Soil on West Side North Approach

Two areas of cracked and distressed asphalt were identified on the South Approach, refer to Figure 12. It is believed that these areas will eventually develop into potholes and it is recommended that these areas be repaired (Maintenance Item M-2).

Figure 12: Areas of Distressed Asphalt on South Approach

On the east side of the South Approach the slope appears to have been supported/reinforced using a no-post barrier, refer to Figure 13. It is recommended that the barrier be removed and a properly anchored support be installed to stabilize the approach fill (Maintenance Item M-3).

On the South Embankment, immediately below Span 1, the north facing slope was observed to have a minor amount of erosion. Due to the size of the South Abutment, the erosion is not a concern but it is recommended that it be monitored during future inspections (Inspection Item I-1).

Figure 13: No-Post Barrier Supporting South Approach Fill

3.2 Abutments

Both the North and South Abutments were sounded using hammers in an effort to locate concrete delaminations. No delaminations were detected although a vertical crack was observed in the south face of the North Abutment wall, refer to Figure 14. The crack was located approximately along the bridge centreline and was observed to be accompanied by efflorescence. Cracks were also observed in the North Abutment wing wall on the east face. These cracks are not considered to be of concern at this time but it is recommended that the condition of the concrete around the cracks be monitored during future inspections (Inspection Item I-2).

The surface of the concrete at both abutments was found to be covered in small amounts of graffiti. It is recommended that the graffiti be painted over as part of regular maintenance (Maintenance Item M-4).

Figure 14: Crack with Efflorescence in North Abutment

3.3 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 the swing staging.

3.3.1 Pier 1

Pier 1 was observed to have numerous cracks in all faces with widths ranging from hairline to a few millimetres. The most significant was a long vertical crack in the east face extending almost the full height of the pier with a maximum width of 6-7 mm near the pier cap. A horizontal construction joint near the top of the pier has also developed into a crack approximately 3 mm wide. The east and west faces of the pier were sounded with hammers using the swing stage access and a number of areas with degraded concrete were identified and marked using red paint; refer to Figure 15 and Figure 16. A large area of severe scaling was identified on the north face of Pier 1 extending approximately 1/3 of the height from the base. The condition of the concrete is not currently a concern but it is recommended that the condition of the concrete in Pier 1 be monitored in the future (Inspection Item I-3). For long-term durability of Pier 1, BC MoT may wish to consider injecting all cracks with epoxy - if and only if the bearings are rehabilitated to restore their original design condition.

Figure 15: Degraded Concrete on East Face of Pier 1

Figure 16: Degraded concrete on West Face of Pier 1

3.3.2 Pier 2

Pier 2 was visually assessed from its base due to the ease of access from the river bed. The lower 1800 mm (6 feet) were sounded using hammers and no hollow areas were identified. The pier is well founded on bed rock.

In addition to sounding the pier concrete, the steel ice shield on the east face of the pier was also sounded and a hollow area was detected on the south edge at the first joint between steel plates from the base of the pier. It is recommended that the concrete in this area be monitored in during future inspections (Inspection Item I-4). For long-term durability of the concrete pier, BC MoT may wish to consider injecting all cracks with epoxy.

3.3.3 Pier 3

Previous inspections have suggested that the bearings located on Pier 3 may have seized which would result in the pier attracting forces for which it had not been designed; this issue is further discussed in Section 3.4. With this in mind, BC MoT requested that special attention be paid to Pier 3 during this inspection. The pier cap, both the north and south faces and portions of the east and west faces were sounded with hammers using access provided from the swing stages, refer to Figure 17.

Figure 17: Sounding the North Face of Pier 3

Numerous cracks were identified in all faces of the pier as well as on the top surface and vertical surfaces of the pier cap, refer to Figure 18. These cracks ranged in width from 1 to 6 mm. Concrete delaminations were identified on the top of the pier cap adjacent to the west bearings, on the north and south sides and on the east and west ends of the pier. In some locations, efflorescence was also observed on the concrete surface at crack locations; refer to Figure 19 and Figure 20. The cracks are not currently a structural concern and no immediate repairs are recommended in the short term but it is recommended that the concrete be monitored during future inspections (Inspection Item I-5). However, for long-term durability of the Pier 3, BC MoT may wish to consider injecting all cracks with epoxy - if and only if the bearings are rehabilitated to restore their original design condition.

Figure 18: Cracks in Pier 3 Pier Cap at West End

Figure 19: Cracks in West Face of Pier 3 with Efflorescence

Figure 20: 6 mm Wide Crack in South Face of Pier 3

3.3.4 Pier 4

The north side of Pier 4 was visually assessed and the lower 1800 mm (6 feet) was sounded from the ground. No delaminations or hollow areas were detected but numerous cracks were observed in the concrete. Horizontal cracks with widths in the order of 2-3 mm were observed at locations believed to be construction joints during original construction. Additionally, cracks were identified in the top of the pier cap as well as in the vertical face of the pier cap approximately at the centreline of the pier, refer to Figure 21 and Figure 22. No remedial actions are recommended but it is recommended that the condition of the concrete be monitored during future inspections (Inspection Item I-6). However, for long-term durability of the concrete pier BC MoT may wish to consider injecting all cracks with epoxy.

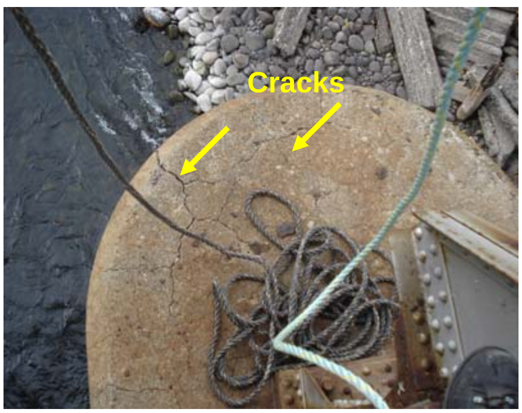


Figure 21: Cracks in Pier Cap of Pier 4

Figure 22: Crack at Centreline of Pier Cap at Pier 4

3.3.5 Pier 5

Pier 5 was sounded on the north, east and west faces using the swing stage access. The north and south faces were also visually assessed from the ground and from the truss during the inspection of Span 5. Various hairline cracks were identified. Four horizontal cracks extending across the width of the pier were observed on the north side that were wider than hairline cracks, refer to Figure 23. It is believed that these locations correspond to construction joints during original construction. One of the horizontal cracks is located at the base of the bearing pedestal for the Span 5 bearings. At the northwest and northeast corners of the pier, this crack terminates at moderate (300 mm x 300 mm) sized spalls. It is recommended that these two spalls be repaired and the horizontal cracks be filled with a product similar to Sikaflex 2C NS (Maintenance Item M-5). In addition, for long-term durability Pier 5, BC MoT may wish to consider injecting all cracks with epoxy - if and only if the bearings are rehabilitated to restore their original design condition. Additionally, graffiti was observed on the north and south faces of the pier and it is recommended that the graffiti be painted over (Maintenance Item M-6).

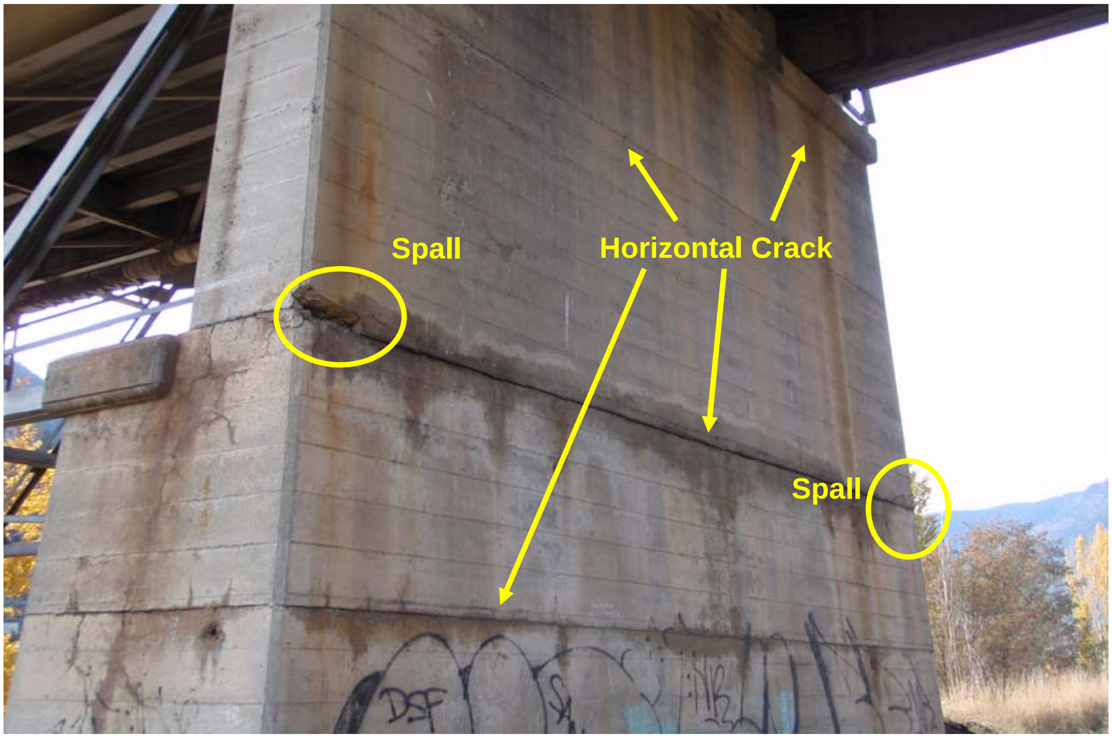


Figure 23: Four Horizontal Cracks and Two Spalls in North Side of Pier 5

3.3.6 Pier 6

Pier 6 was visually assessed and the bottom 1800 mm (6 feet) was sounded using a hammer. No delaminations or hollow areas were identified although a number of cracks were observed. In addition, graffiti was observed on the north and south faces of the pier. It is recommended that the graffiti be painted over (Maintenance Item M-7). Cracks were observed in the face of the bearing pedestal on top of Pier 6 that supports the longitudinal stringer in the girder spans. It is recommended that all cracks be monitored during future inspections (Inspection Item I-7).

3.4 Bearings

Each of the seven spans are 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.

Two types of sliding bearings have been used on the Old Spences Bridge. For the girder spans and the three shorter truss 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. In many locations pack rust was observed in the gap between the two plates, refer to Figure 24. 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. It is recommended that rehabilitation options include repairing or replacing the sliding bearings in the girder spans and the shorter truss spans (Rehabilitation Item R-1).

Figure 24: Pack Rust between Bearing Plates

Figure 25: Roller Bearings at Pier 3

The sliding 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 25. 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, refer to Figure 26.

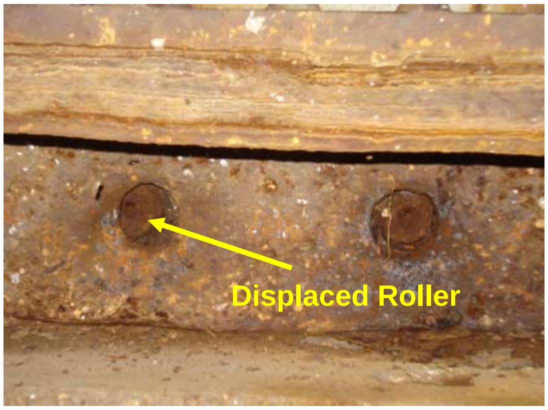


Figure 26: Roller No Longer in Guide Plate in Bearings at Pier 3

Figure 27: Tight Washer at Anchor Bolt

All four of the sliding bearings 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 assembly. No obvious signs of longitudinal movement were observed and it is believed 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, refer to Figure 27.

Many of the anchor bolts were found to be out of plumb (i.e. they are no longer vertical), refer to Figure 28. 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 inclined from the vertical. 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, refer to Figure 29. 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.

Figure 28: Inclined Anchor Bolts at Pier 3

Since it is believed that the bearings are seized, then it is likely that Pier 3 is being subjected to loads for which it was not originally designed. It is recommended that Pier 3 be evaluated for the effects of the seized bearings (Evaluation Item E-1) and that rehabilitation options include repairing or replacing the sliding bearings at Pier 3 (Rehabilitation Item R-2).

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 30.

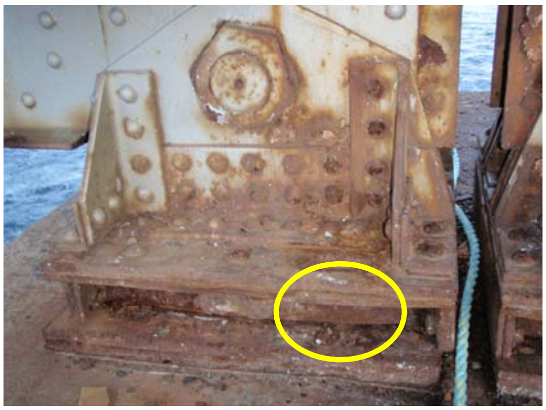


Figure 29: Anchor Bolt Sheared off at Span 4 Southeast Bearing

Figure 30: Evidence of Rotation at Keeper Nut

3.5 Truss Components

3.5.1 Structural Steel Coating

As part of the 2009 Inspection of the Old Spences Bridge, Trans Canada Coating Consultants Ltd. (TC3) were retained to collect data on the condition of the coating, provide an estimated remaining life for the existing structural steel coating, and recommend economical treatments. TC3 found that in order "to gain useful life for the bridge the corrosion must be slowed or stopped". A copy of the TC3's report is included in Appendix D.

Numerous areas of widespread and localized areas of coating failure were observed throughout the structure. The greatest concentration of these areas is on the structural steel in the vicinity of the deck joints. Specifically, the floorbeams, stringers, sway bracing, areas on the top chord lateral bracing and the top chord cover plates were found with the largest areas of coating failure. Figure 31 and Figure 32 show the typical failures of the protective coating near the centreline of the bridge for the deck joints and expansion joints respectively. The condition of the coating at the outer stringers is typically worse as shown in Figure 33 and Figure 34.

Figure 31: Typical Coating Failure near Centreline of Bridge at Deck Joint

Figure 32: Typical Coating Failure near Centreline of Bridge at Expansion Joint

Figure 33: Typical Coating Failure at Outer Stringer on West Side at Floorbeam

Figure 34: Typical Coating Failure at Outer Stringer on East Side at Floorbeam

It was also noticed that coating failures typically occur on most members beneath the short deck drains that release deck run-off just below deck level. Degradation was observed to the face of the top chord at the deck drain and to the bottom chords. The smaller trusses have greater coating failure to the bottom chords since they are closer to the drains.

Figure 35: Vertical Area of Coating Failure at Top Chord Connection below Drain

Figure 36: Widespread Coating Failure at Top Chord Connection below Drain

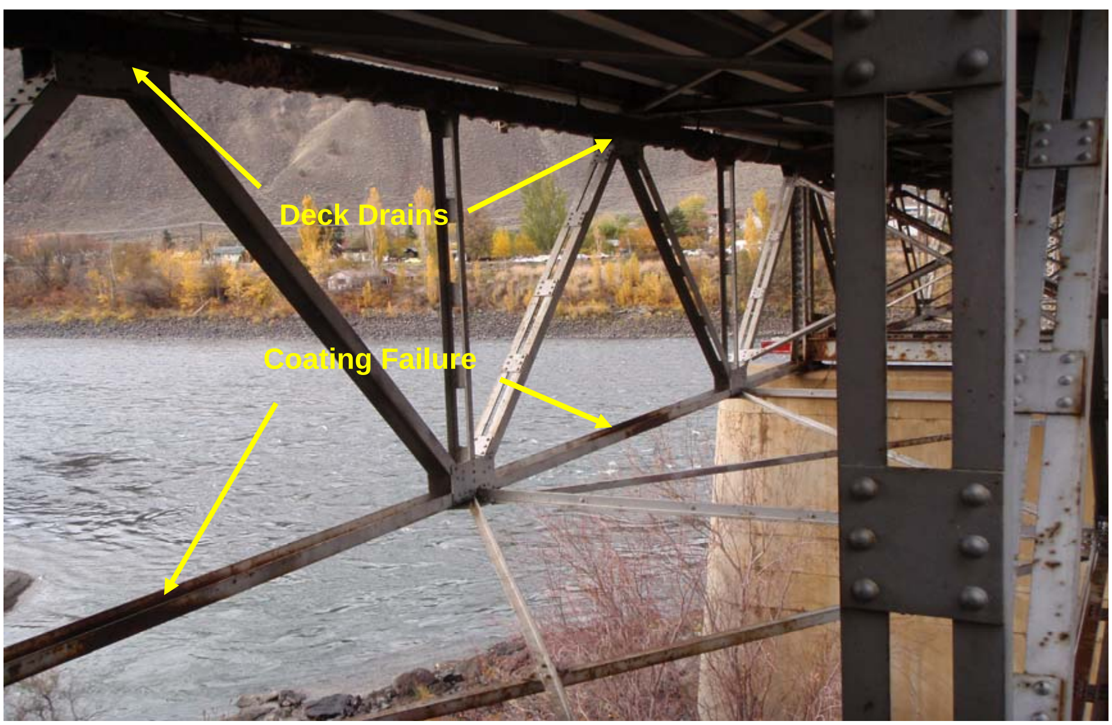


Figure 37: Coating Failure of Bottom Chord from Deck Drain Above

It is recommended that, in the short term, BC MoT consider tendering a contract to apply an easy to use, surface tolerant material, such as Termarust (or equivalent), to the corroding areas to drastically slow or stop additional corrosion (Rehabilitation Item R-3). In the longer term, it is recommended that BC MoT tender a contract to strip the bridge to bare metal and recoat the entire structure with a three coat zinc/epoxy/urethane system (or equivalent) (Rehabilitation Item R-4).

3.5.2 Top Chord

The top chords were typically found to be in fair to good condition with some isolated areas in very poor condition with significant corrosion and even some perforations in the structural steel due to advanced corrosion.

A small number of isolated areas, typically at the east drain locations, were identified with coating failure and light to moderate surface corrosion. No significant section loss was found in the chord members themselves with the exception of one location in the web of the east top chord at Panel Point 3 in Span 4, refer to Figure 38. It is recommended that this localized area of section loss be evaluated to determine if repairs are required (Evaluation Item E-2). As stated in Section 3.5.1, it is also recommended that recoating the corroding areas on the top chords of the truss spans be included (Rehabilitation Item R-3).

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.

The top chords of simply supported trusses, like the truss spans on the Old Spences Bridge, are compression members and batten plates are used to brace the chord members against local buckling by reducing their effective lengths; cover plates are used to add to the cross sectional area that resists the compressive forces. Because the locations of section loss are adjacent to the floorbeams, 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. However, it is recommended that the corrosion in these areas be slowed or stopped if possible. This can be accomplished by including these areas in Rehabilitation Item R-3.

Figure 38: Localized Area of Section Loss in East Top Chord Web at Panel Point 3 – Span 4

Due to significant localized corrosion, a hole has formed in the top chord cover plate adjacent to the east end of Floorbeam 6 in Span 2, refer to Figure 39. It is recommended that the effect of this hole in the top chord cover plate be evaluated (Evaluation Item E-3) and, unless the evaluation determines that the plate requires replacement, that the plate be included in Rehabilitation Item R-3.

Figure 39: Hole in Cover Plate on Top Chord at Floorbeam 6 – Span 2

3.5.3 Bottom Chord

Unlike the top chord, which is partially sheltered by the bridge deck, the bottom chord is exposed to the elements. In addition to being exposed to the elements, sections of the bottom chord are located below the deck drains which concentrate the run-off from the bridge deck directly onto portions of the chord members.

The majority of the bottom chord members are in fair condition along their length although numerous areas of section loss were identified during the inspection. Significant lengths of the bottom chord members exhibit coating failure with light surface corrosion and it is recommended that they be included in a recoating program that includes the entire bridge structure (Rehabilitation Item R-4).

In Spans 1, 2 and 5, areas of section loss were found on the vertical leg of the angles directly below the deck drain locations. The most significant areas with section loss are listed in Table 1. It is recommended that these members be evaluated to determine the effects of the observed section loss on the load-carrying capacity of the bottom chord (Evaluation Item E-4).

Table 1: Most Severe Bottom Chord Members with Observed Section Loss

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

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, refer to Figure 40 and Figure 41. 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. It is recommended that the capacity of the bottom chord members be evaluated to determine if the existing reinforcing system remains adequate (Evaluation Item E-5). Additionally, a localized area with approximately 8 mm of section loss in the top flange of the bottom chord channel was identified at Panel Point 4 on the east side of Span 4. This corresponds to a small area of the top flange with roughly 50% section loss of the total thickness as the un-corroded thickness of the top flange is 16 mm (5/8"). It is recommended that this area be evaluated to determine the effect of this section loss on the structural capacity of the member and establish whether immediate repairs are required (Evaluation Item E-6).

Many of the batten plates along the bottom chord members were found to have corrosion, corrosion product build-up and section loss, ranging from areas of light to complete section loss. This is not a concern because the bottom chord members are tension members and do not rely on batten plates for strength or stability. However, there is a concern that corrosion in the batten plates could progress into the chord members themselves and it is recommended that the batten plates be cleaned and Ministry approved coating be applied (Rehabilitation Item R-5). At that time, BC MoT may elect to replace the batten plates exhibiting areas of complete section loss as they do assist in providing access to the bottom chord during bridge inspections. It is estimated that approximately 50% of the bottom chord batten plates in Spans 3 and 4 require cleaning and approximately 15% contain perforations due to corrosion.

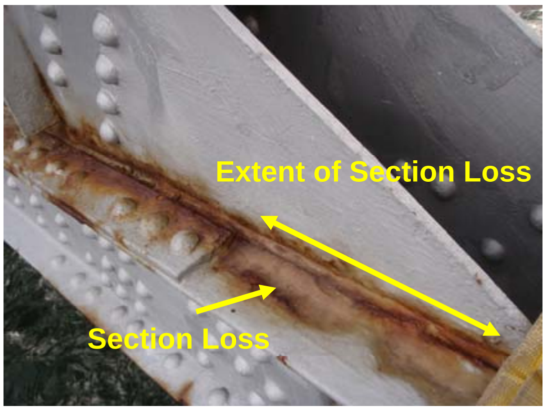


Figure 40: Section Loss in Top Flange of Bottom Chord Channel

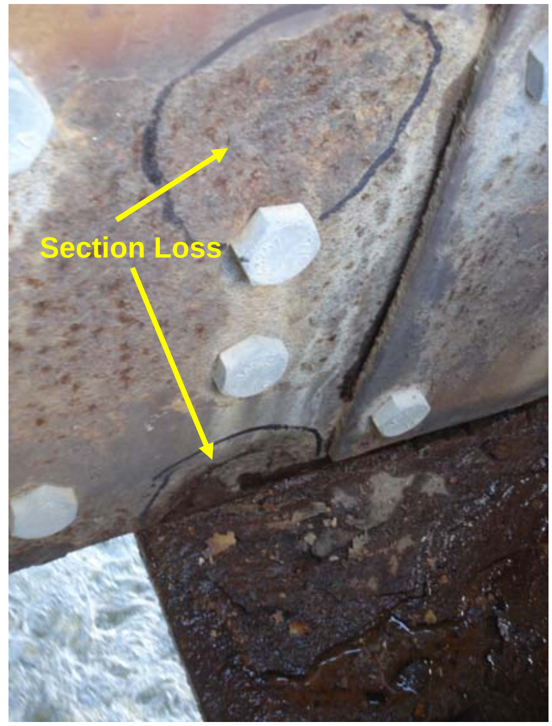


Figure 41: Section Loss in Back of Web of Channel

3.5.4 Verticals

The vertical members in the truss spans were found to be in good condition with a small number of isolated areas of coating failure and surface corrosion observed. The connection between the verticals and the top chord was typically found to be in good condition but 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; refer to Figure 42. Additionally, pack rust was observed in the joint between the gusset plate and the back side of the channel webs; refer to Figure 43. During the inspection, due to the presence of the gusset plate on the inboard face of the chord and the added plate on the outboard face of the chord, it was not possible to establish whether this pack rust resulted from corrosion and section loss of the bottom chord web member.

It is recommended that the gusset plates with significant section loss, perforations and pack rust be replaced and that all gusset plates with corrosion or minor section loss be cleaned and recoated with a Ministry approved coating (Rehabilitation Item R-6). A list of locations in Spans 3 and 4 where the gusset plates require rehabilitation is included in Table 2 below. There is no immediate structural concern for these vertical member gusset plates, even though some have significant perforations, since these gusset plates all connect zero force vertical truss members to the bottom chord of the truss (i.e., they carry very little load).

Table 2: Vertical Member Gusset Plates with Observed Perforations/Section Loss or Rust Jacking

Span	Panel Points
3	PP1,PP3,PP5,PP7,PP9
4	PP1,PP3,PP5,PP7,PP9

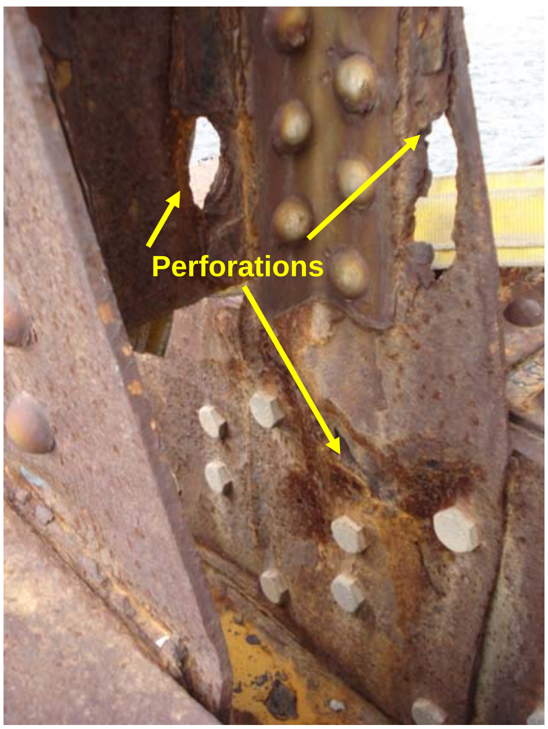


Figure 42: Perforations in Gusset Plate due to Corrosion at L1-U1 in Span 3

Figure 43: Rust Jacking Between Gusset Plate and Chord Member and Hole in Batten Plate

3.5.5 Diagonals

The diagonal members were found to be generally in good condition with only limited areas of coating failure and light surface corrosion observed.

A small dent was observed in member U1-L2 in the west truss of Span 3, refer to Figure 44. It is believed that this dent is a result of an impact during original construction. The member is primarily a tension member and therefore no remedial actions are recommended with regards to the dent.

A rivet was missing in Span 4 in the connection between member L6-U7 and the bottom chord, refer to Figure 45. It is recommended that a fully tensioned bolt be installed in the empty rivet hole (Maintenance Item M-8).

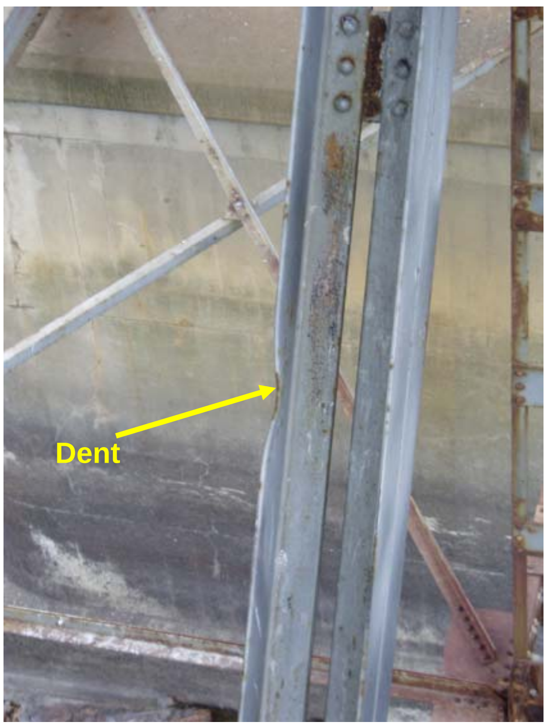


Figure 44: Dent in Diagonal U1-L2 Span 3 – West Truss

Figure 45: Missing Rivet in Diagonal L6-U7 – West Truss

Multiple perforations, listed in Table 3 below, were identified in the batten plates at the lower end of the diagonals and many of these batten plates have been previously repainted. The perforations are not a concern at this time due to the location of the batten plates but it is recommended that the plates be monitored during future inspections (Inspection Item I-8).

Table 3: Locations of Perforated Batten Plates

Span	Member	East/West
3	U1-L2	West
4	L2-U3	West
4	U3-L4	West

A localized area of minor section loss was identified at the base of the web near the top of member L4-U5 on the west side of Span 3, refer to Figure 46. It is recommended that this area be recoated as part of Rehabilitation Item R-3. Additionally, a portion of member L8-U9 on the west side of Span 4 was observed with coating failure and surface corrosion, refer to Figure 47. The corrosion is not severe and is unlikely to result in section loss in the near future. Therefore, it is also recommended that this member be cleaned and recoated during a recoating program for the entire structure (Rehabilitation Item R-4).

Figure 46: Localized Section Loss at Base of Web in Member L4-U5 Span 3 West

Figure 47: Coating Failure on Member L8-U9 Span 4 West

3.5.6 Bottom Chord Panel Point

A typical corrosion pattern was identified at the even numbered bottom chord panel points in Spans 3 and 4. The gusset plates that connect the vertical and diagonal members to the bottom chord were observed to have areas of section loss in the gusset plates along the level of the bottom chord top flange, refer to Figure 48. It is believed that the section loss was caused by the accumulation of debris. However, no debris was observed in these locations during the inspections suggesting that it has been removed and it appears that many of the locations have been repainted which will serve to retard corrosion. In the majority of locations, the section loss was only noted on the interior gusset plate although areas of section loss were noted in a small number of exterior gusset plates as well.

It is recommended that the gusset plates be evaluated in order to determine the impact of the observed corrosion and to determine possible rehabilitation options (Evaluation Item E-7).

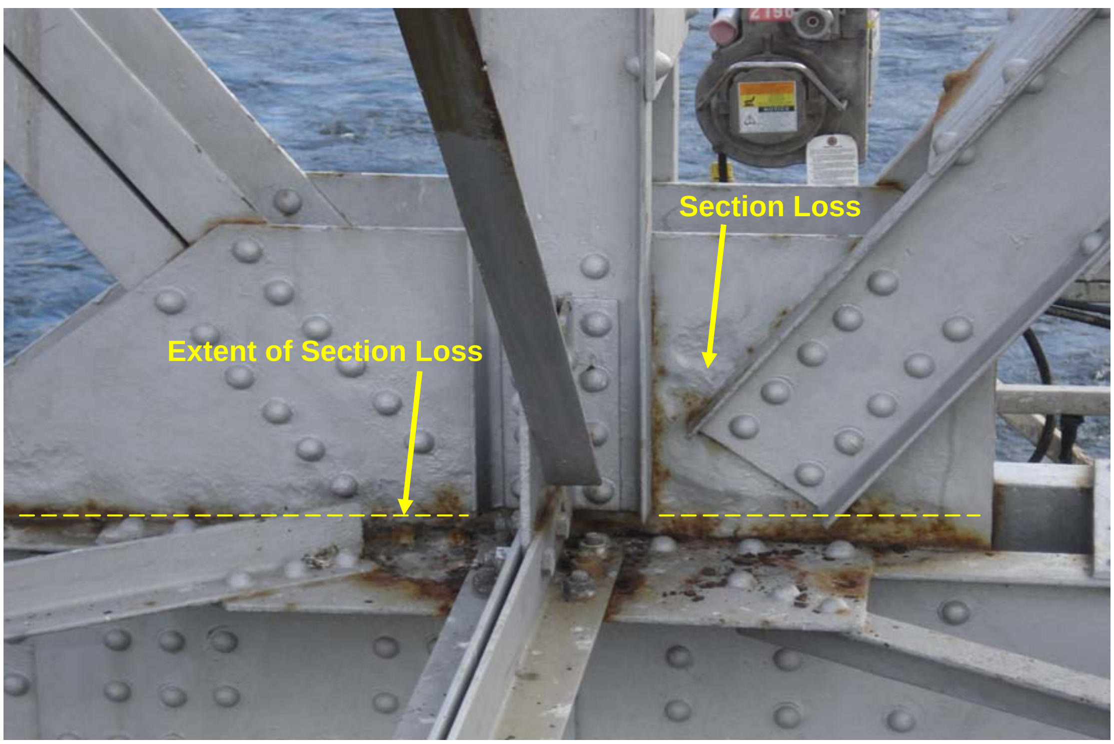


Figure 48: Section Loss along Gusset Plate

3.5.7 Lateral Bracing

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 no significant 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.

The bottom chord lateral bracing connections were found to be in worse condition than the top chord bracing connections due to their exposure to the elements. Numerous areas of section loss were identified on the bottom chord bracing connections and multiple holes were found, refer to Figure 49 and Figure 50. The bottom chord lateral bracing connections appear to be functioning adequately in their current condition and it is not believed that a widespread repair program is warranted at this time. It is recommended that the connections be cleaned, a Ministry approved coating applied and that they be monitored on an annual basis and replaced or repaired as necessary (Rehabilitation Item R-7 and Inspection Item I-9).

Figure 49: Areas of Section Loss in Gusset Plate

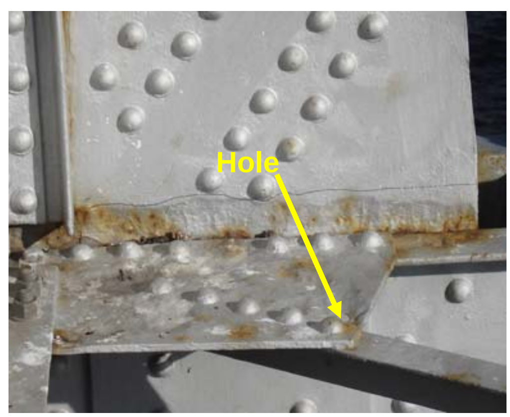


Figure 50: Hole in Gusset Plate

The top chord bracing connections are riveted to the top flange of the top chord at the panel points. The connection plates were typically found to have complete coating failure on their surfaces with light surface corrosion. It is recommended that these plates be recoated as part of Rehabilitation Item R-3.

3.5.8 Jacking Beams

Rolled I-shaped beams are provided at the bearing locations and provide a transverse strut between the bearings. These beams are intended to serve as jacking points in the event that the bearings need to be replaced or serviced. Many of the jacking beams were found to have significant corrosion on their webs and flanges and a hole was observed in the web of the jacking beam between the north bearings of Span 1, as seen in Figure 51. It is believed that the jacking beams would be unsuitable for carrying the load required to replace bearings in their current condition. The jacking beams are not believed to be required as primary load carrying members and no repairs are recommended at the current time. However, if BC MoT chooses to replace the bearings on the Old Spences Bridge, it is recommended that the capacity of the jacking beams be evaluated considering their current condition and that they be reinforced as required (Rehabilitation Item R-8). A list of all jacking beams and a description of their observed condition is included in Table 4 below.

Table 4: Condition of Jacking Beams

Location	Condition
Span 1 L0	Widespread coating failure on web with light surface corrosion.
Span 1 L6	Hole in jacking beam web. Numerous isolated areas of dishing (section loss) in top flange.
Span 2 L0	Jacking beam top flange has localized 2-3 mm section loss with some very local locations up to 4-5 mm section loss. Jacking beam shows signs of web buckling (possibly due to impact during original construction) at mid span of the beam.
Span 3 L0	Jacking beam has areas of localized corrosion product build-up (up to 10 mm deep) on top of top flange. Corrosion product build-up identified on web (5-10 mm thick) Localized section loss is approximately 10 mm in 1 or 2 places, typically 6 mm in 3 - 4 other locations. Diameter of section loss is approximately 50-75 mm.
Span 3 L10	Fair Condition. Coating on web mostly intact. Localized areas of corrosion product build-up on top flange.

Location	Condition
Span 4 L0	4 mm deep dishing in top flange of jacking beam 100 mm x 100 mm.
Span 4 L10	Localized areas of corrosion on jacking beam.
Span 5 L8	Localized areas of corrosion product build-up on top flange of jacking beam and on bottom flange.

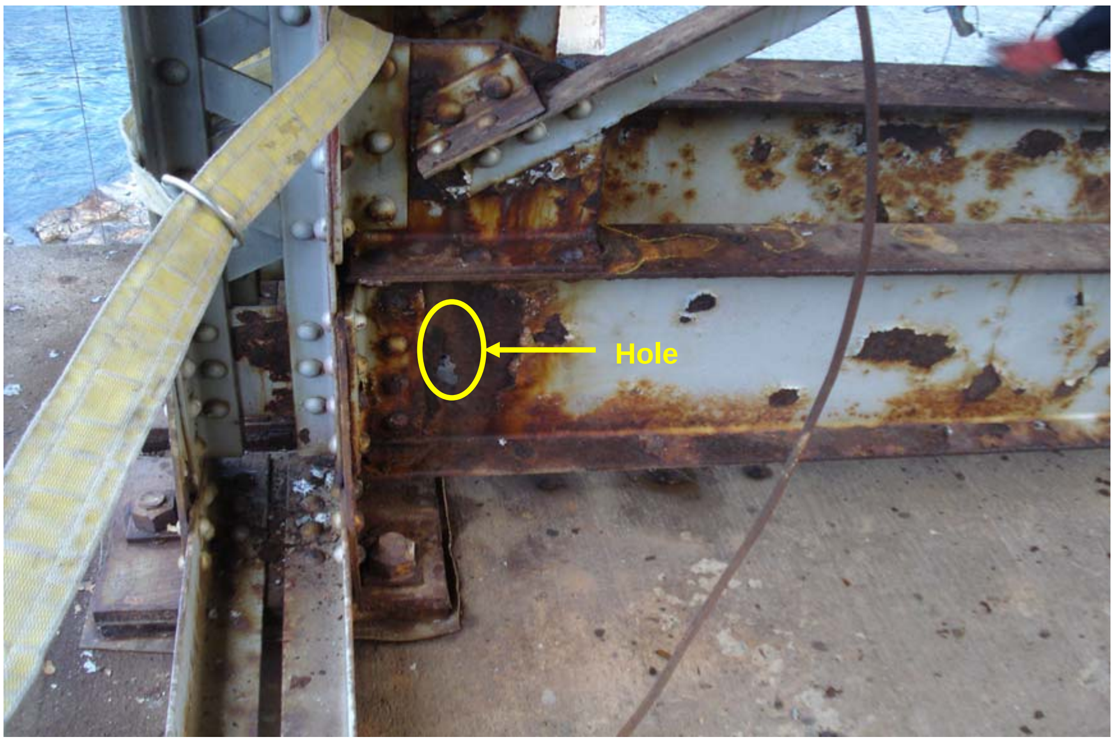


Figure 51: Hole in Span 1 Jacking Beam Web at Pier 1

3.5.9 Sway Bracing

Many of the sway bracing members were observed to be in poor condition which is believed to be due to the fact that they are located directly below the deck joints. Coating failure and surface corrosion were widespread on both the members and gusset plates and numerous holes were identified in members and gusset plates, refer to Figure 52 and Figure 53. Table 5 lists the sway bracing locations where holes were observed in the members. It is recommended that these bracing members be replaced (Rehabilitation Item R-9) and that the remaining bracing members be recoated when the entire structure is recoated (Rehabilitation Item R-4).

Table 5: Locations of Sway Brace Members with Perforations

Span	Panel Point	Member
3	4	Mid-height transverse strut (structural angle)
3	6	Mid-height transverse strut (structural angle)
3	10	Mid-height transverse strut (structural angle)
4	0	Mid-height transverse strut (structural angle)

Figure 52: Holes in Gusset Plate at Sway Brace at Panel Point 10 – Span 4

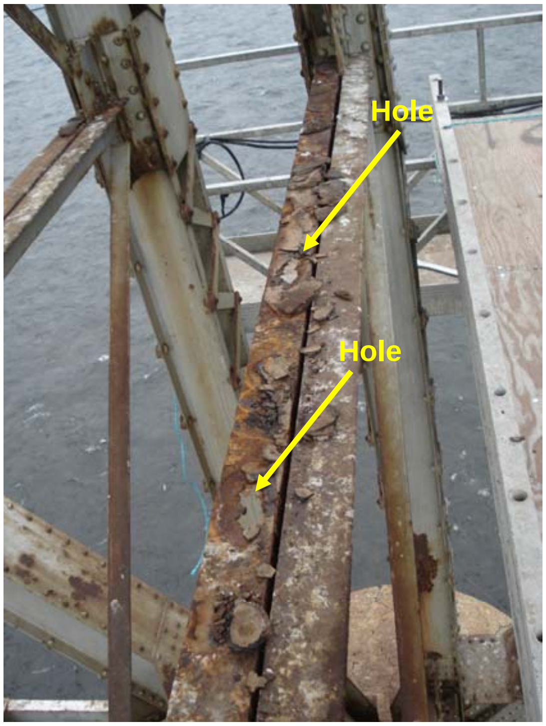


Figure 53: Holes in Brace Member at Panel Point 10 – Span 3

It is also recommended that the sway brace gusset plates with holes and significant section loss be repaired or replaced when the perforated bracing members are replaced (Rehabilitation Item R-9). Table 6, lists the gusset plates that likely require repairs or replacement, based on the observations made during this inspection.

Table 6: Sway Bracing Gusset Plates with Observed Section Loss and Perforations

Span	Panel Point	Member	Repair/Replace
3	PP2 – west	Mid-height	Repair
3	PP0 – east	Base	Replace
3	PP4 – east	Base	Repair
3	PP6 – west	Mid-height	Repair
3	PP6 - west	Base	Repair
4	PP4 – west	Mid-height	Repair
4	PP4 – east	Base	Repair
4	PP8 - west	Base	Repair
4	PP6 - west	Base	Replace
4	PP10 - west	Mid-height	Replace
4	PP10 - east	Mid – height	Repair

3.5.10 Deck System

The concrete bridge deck and the sidewalk were visually inspected and chain dragged to locate voids, delaminations and spalls. The underside of the concrete deck was also visually assessed and sounded with hammers.

Numerous deficiencies were identified on both the top and bottom surfaces of the concrete deck and are presented below. Due to the numerous deficiencies observed, it is recommended that an evaluation be carried out on the existing bridge deck and rehabilitation options be developed that include the feasibility of replacing the existing deck in part or in its entirety (Evaluation Item E-8 and Rehabilitation Item R-10).

Both concrete curbs along the deck were found to be in poor condition with cracks and spalling concrete observed in multiple areas, refer to Figure 54. Evidence of previous repairs were observed along the west curb but it appears that these repairs are not performing well, refer to Figure 55. It is recommended that an evaluation and rehabilitation of the deck include repairing the concrete curbs (Evaluation Item E-8 and Rehabilitation Item R-10.

Figure 54: Crack in West Curb Face

Figure 55: Curb Deterioration at Previous Repair

No delaminations were detected while chain dragging the top surface of the roadway deck however a crack pattern similar to a spider web was observed at the four corners of each panel, refer to Figure 56. It is recommended that the deck be pressure washed and a silane sealer applied to the areas exhibiting cracks (Maintenance Item M-9).

Figure 56: Crack Pattern at Corner of Deck Panel

In contrast to the roadway deck, numerous delaminations were detected while chain dragging the sidewalk. The areas with delaminations and the areas with spalls were marked with red paint and are concentrated towards the south end of the bridge deck, refer to Figure 57 and Figure 58.

Figure 57: Voids and Spalls Noted in Sidewalk

Figure 58: Voids Detected in Sidewalk

While the top side of the concrete deck appeared to generally be in fair condition, widespread honeycombing was observed on the underside of the deck which is a result of poor consolidation of the concrete during original construction, refer to Figure 59. Additionally, numerous cracks, delaminations and spalls were noted on the underside of the bridge with exposed rebar visible at a number of locations; refer to Figure 60 and Figure 61. A plan view of the bridge showing the observed areas of delaminations and spalls has been provided in Appendix E. It is recommended that the evaluation of the existing deck include repairs to the delaminations and spalls observed on the underside of the concrete deck (Evaluation Item E-8 and Rehabilitation Item R-10).

The condition of the structural steel below deck can be directly attributed to the deck joints at each floorbeam. At the time of the inspection the joints were filled with a mastic compound but previous inspection reports indicate that the joints have not always been filled. It is not known when the mastic compound was added to the joints but it appeared to be performing adequately at the time of the inspection. It is recommended that the performance of the mastic joints be monitored during future inspections (Inspection Item I-10).

Figure 59: Typical Honeycombing in Underside of Concrete Deck

Figure 60: Deck Delamination above Pier 6

Figure 61: Exposed Rebar in Span 6

3.5.11 Stringers

The longitudinal stringer system has been broken into two categories; deck stringers (DS) and sidewalk stringers (SS); refer to Figure 8 and Figure 9.

3.5.11.1 Deck Stringers

For the purpose of presenting the inspection findings, the deck stringers have been 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 likely due to their increased exposure from each side of the bridge and, in the case of DS1, the location of the curb above. Numerous widespread areas of coating failure and surface corrosion were observed along these exterior stringers with localized areas of section loss in the web identified, refer to Figure 62. Significant amounts of corrosion product build-up and section loss were also identified on the underside of the top flange and on the top and bottom surfaces of the bottom flange. It is recommended that cleaning and recoating the deck stringers be included in Rehabilitation Item R-3.

Figure 62: Coating Failure and Surface Corrosion on DS1

The majority of interior stringers were observed to have coating failure and surface corrosion on the webs and flanges at their ends, refer to Figure 63. The corrosion was typically found to extend approximately 100-200 mm (4-8”) from the end of the stringer and was found to vary in severity with the exterior stringers exhibiting more advanced corrosion and section loss and the interior stringers exhibiting minor corrosion.

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 have been marked for repair but the repairs have not been completed, refer to Figure 64 and Figure 65. Based on the corrosion and section loss observed in the stringers, it is recommended that the capacity of the stringers be evaluated (Evaluation Item E-9). Considering the variability in the condition of the stringers, the following evaluation criteria is recommended for the longitudinal deck stringers:

Calculate the allowable amount of corrosion for the stringers based on the following assumptions:

- Calculate the permissible level of section loss for the top flange assuming no section loss in the web or in the bottom flange. This criteria applies to the stringers at mid-span; and
- Calculate the permissible level of section loss for the web assuming no section loss in the top or bottom flange. This criteria applies to the ends of the stringers.

The results of the evaluation, in conjunction with the inspection observations, can be used to develop rehabilitation options for the deck stringers.

Regardless of whether the stringers are found to have sufficient load carrying capacity or if repairs are required, it is necessary to halt continued corrosion and section loss (Rehabilitation Item R-3).

Figure 63: Typical Corrosion Pattern on Stringers

Figure 64: Previous Repair on Exterior Stringer

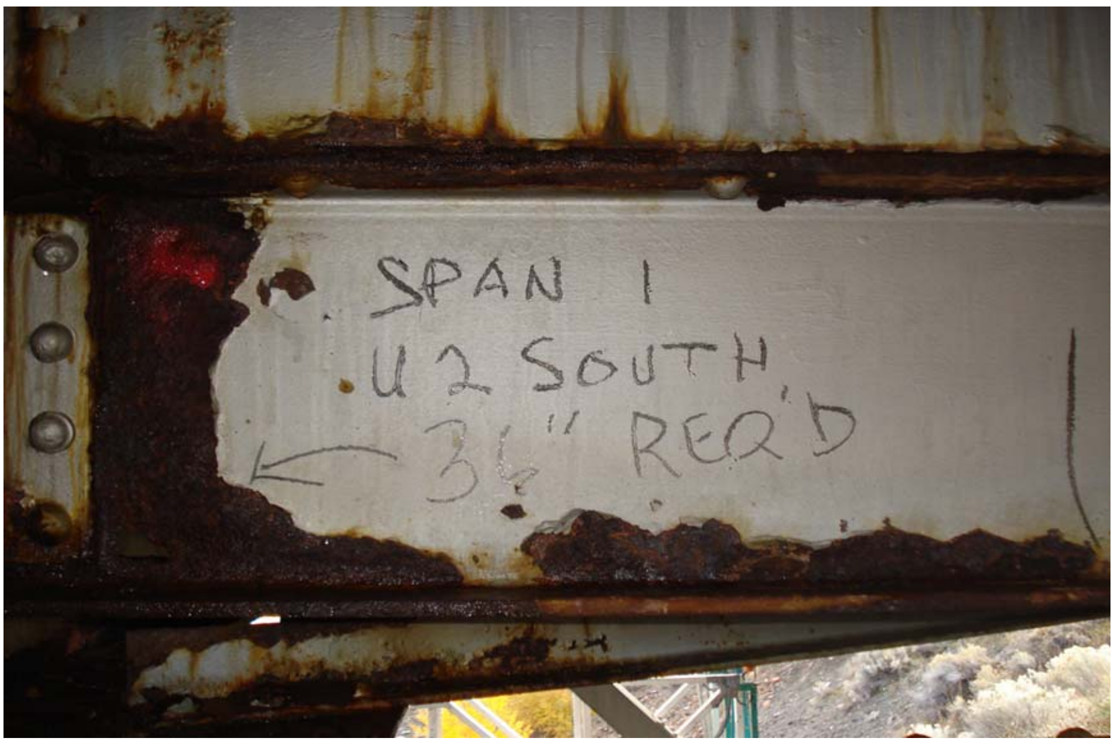


Figure 65: Stringer Marked for Repair but not yet Repaired

During the inspection it was also observed that many of the stringers are no longer in contact with the underside of the concrete slab. This is discussed in Section 3.5.12.

3.5.11.2 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.

The backside of the web of SS3 was typically observed to be covered in surface corrosion over large areas along its length with minor to moderate section loss identified in isolated areas occasionally identified in the web near mid-span. Major section loss and perforations were observed in multiple stringers at the connection to the floorbeams, refer to Figure 66 and Figure 67. The stringer is supported along its length by DS1 and the localized section loss is not considered to be an immediate structural concern. However, it is recommended that the stringers be included in Rehabilitation Item R- 3.

Figure 66: Holes in Web of SS3 at Floorbeam 0, Span 2

Figure 67: Complete Section Loss in Web of Channel at Floorbeam 10, Span 3

Additionally, pack rust was 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, refer to Figure 68. It is recommended that these areas be included in Rehabilitation Item R-3.

Figure 68: Pack Rust Causing Deformation of Top Flange

Multiple locations, typically at SS2, were found where the stringer webs were not vertical suggesting that they may be buckling or that a horizontal load may have acted on them at some point in time. This pattern is difficult to explain given the current condition of the deck and stringers because in many cases, the stringers that are not vertical are also not in contact with the underside of the concrete deck. The fact that the webs are not vertical is possibly due to lateral torsional buckling due to a lack of support to the compression flange.

It is believed that, although long sections of the stringers are not being loaded by the deck, localized areas of pack rust are attracting load into the stringers along their length. The absence of support to the compression flange of the stringers combined with loading from the deck is possibly the causing lateral torsional buckling of the stringers. It is recommended that an evaluation of the stringers include the effect of reduced lateral support to the compression flange (Evaluation Item E-9).

Figure 69: SS2 Web Not Vertical between U0 and U1 in Span 4

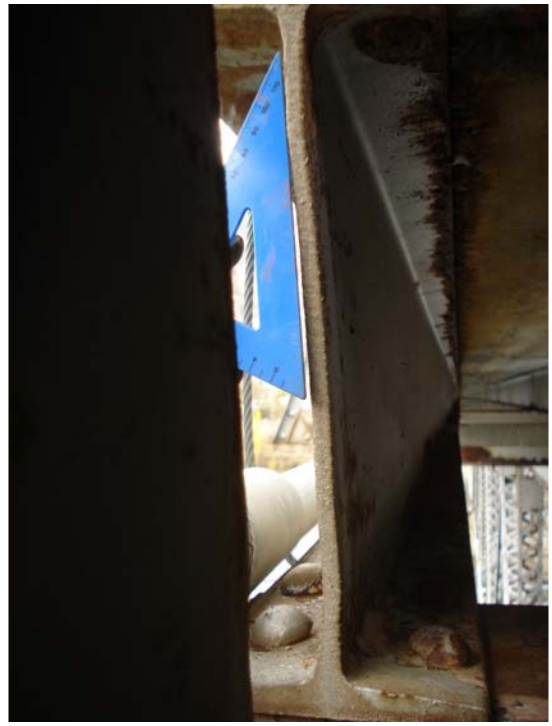


Figure 70: SS1 Web Not Vertical between U1 and U2 in Span 3

3.5.12 Floorbeams

The inspection findings for the floorbeams can be grouped as the condition of the top flange of the floorbeam and the condition of the webs at each end.

The deck joints located directly above each floorbeam have leaked over the years resulting in the formation of pack rust on the top flange of the majority of floorbeams. Due to the accumulation of pack rust between the top flange and the underside of the concrete deck, rust jacking has occurred, lifting the concrete deck off the stringers. In many locations, the thickness of the pack rust on the top flange is significant and, in one location, was measured to be equivalent to the thickness of the top flange. The amount of pack rust observed suggests that moderate section loss has likely occurred in the top flange of the floorbeams and it is recommended that the floorbeams be evaluated to ensure that they have adequate structural capacity (Evaluation Item E-10 see below).

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. It is recommended that the loss of section of the flange be evaluated. (Evaluation Item E-10)

Areas of section loss were also identified in the web at the ends of the floorbeams. On the west end, the areas were typically concentrated along the lower 50-100 mm of the web in the cantilever section. The amount of section loss in these areas was typically 1-3 mm. Similar amounts of section loss were also observed on the faces of the web between stringers DS1 and DS2. At the east end of the floorbeams, the webs of the floorbeams in the vicinity of the top chord were observed to have varying degrees of section loss. Complete section loss was observed in the end of floorbeam 4 in Span 4 and in the web of floorbeam 8 in Span 4, refer to Figure 71 and Figure 72. It is recommended that these areas be repaired (Rehabilitation Item R-11).

Figure 71: Hole in Web of Floorbeam 4 Span 4

Figure 72: Hole in Web of Floorbeam 8 Span 4

Based on the observations made during this inspection and considering the variability in the extent of corrosion observed on the floorbeams, the following evaluation criteria are recommended (Evaluation Item E-10):

- Calculate the permissible level of section loss for the top flange assuming no section loss in the web or in the bottom flange. This criteria applies to the floorbeams at mid-span;
- Calculate the permissible level of section loss for the web assuming no section loss in the top or bottom flange. This criteria applies to the ends of the floorbeams; and

- Calculate the allowable amount of section loss for the top flange of the floorbeam over the top chord assuming 2 mm section loss over the height of the web and no section loss in the bottom flange. This criteria applies to the sidewalk cantilever portion of the floorbeams (negative moment region).

The results of the evaluation, in conjunction with the inspection observations, can be used to develop rehabilitation options for the floorbeams.

As previously mentioned in Section 3.5.11.1, the formation of pack rust between the floorbeam top flange and the underside of the concrete deck has resulted in rust jacking of the deck such that there is a gap between the concrete deck and the longitudinal stringers (i.e. the deck is resting only on the floorbeams and not on the stringers). This has altered the behaviour of the deck which was originally designed to span transversely and is now spanning longitudinally. If the deck does not have sufficient reinforcing to span longitudinally it will crack and carry the load in any way possible. It is believed that the deck is now behaving as if it is in 2-way bending, resulting in the crack pattern observed in the top of the deck. It is recommended that rehabilitation options be developed to address the rust jacking on the floorbeams (Rehabilitation Item R-12).

3.5.13 Deck Drains

In the truss spans, the deck drains extend only a short distance below deck level as shown in Figure 73. The short deck drains release deck run-off directly onto the bottom chord and the run-off also sprays onto the top chord gussets and verticals. Over the years spray from the run-off and the direct drainage onto the structural steel has caused significant localized corrosion and section loss of the bottom chord, of the truss verticals and their gussets at the bottom chord, and of the top chord (refer to sections 3.5.1, 3.5.2, 3.5.3 and 3.5.4).

It is recommended that the deck drains be extended to ensure that the deck run-off drains below the bottom chord to prevent future localized corrosion and section loss (Rehabilitation Item R-13).

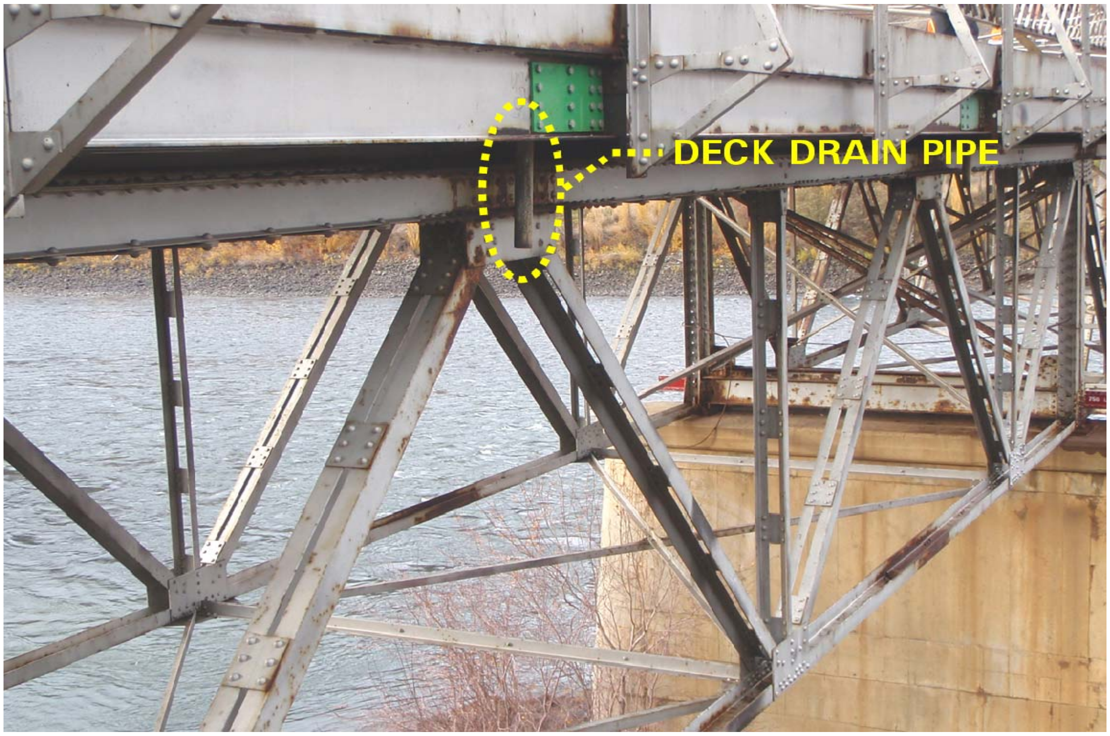


Figure 73: Typical Deck Drain Pipe in Truss Span

3.6 Girder Spans

The girder spans were found to generally be in good condition although localized areas of coating failure were observed along the girders. The majority of the surfaces in Span 6, which crosses the CN rail lines, were found to be covered in soot from passing locomotives. The soot does not appear to have had any deleterious effects on the concrete or the steel and no remedial actions are recommended. At the location of the north most floorbeam in Span 6, the west edge of the bottom flange of the west girder has been impacted by train cargo and is bent upwards, refer to Figure 74. This may also be due to damage during original construction. A close visual inspection of the flange did not find any cracks in the steel although a small gouge was found. This impact damage is not an immediate structural concern with regards to the stability of the span but it is recommended that the girder be evaluated to determine if the load carrying capacity is affected (Evaluation Item E-11).

Figure 74: Impact Damage to West Girder Span 6

During the inspection it was found that the web of the east edge girder in Span 6 appears to have buckled slightly at the bearing point on Pier 6, refer to Figure 75. There are no bearing stiffeners at this location and it is believed that the rust jacking on top of the edge girders may have altered the load path of the span and is attracting additional load to the edge girders. This belief is further supported by observations made with regard to the longitudinal stringers in Span 6 and 7 and the crack pattern observed in the deck. When originally constructed, each of the longitudinal stringers was supported on a bearing plate at each end. This system transferred a portion of the vertical loads from the deck into the stringers and then down into the piers. However, during the inspection it was observed that the many of the stringer bearing plates had a gap between the underside of the plate and the concrete surface. This gap prevents the stringers from carrying a portion of the vertical load and transfers all vertical loads into the edge girders. It is recommended that shim plates be installed under the stringer bearing plates to restore the original load path and that bearing stiffeners be installed on each of the edge girders at their bearing points (Rehabilitation Item R-14). Even if shim plates are installed, it is recommended that Evaluation Item E-11 include the increased loads in the edge girders resulting from the rust jacking.

Figure 75: Web of East Girder in Span 6 at Pier 6 – Possibly Buckled

3.7 Railings

The railings along each side of the Old Spences Bridge are composed of panels of steel lacing with channels as the main longitudinal members. These panels are attached to support brackets that are connected to the ends of each floorbeam and to the exterior stringers at the midpoint of each bay.

The railings were observed to be damaged in numerous areas although the damage is not considered to have a significant affect on its capacity. They were also observed to be bowed between each support bracket, as seen in Figure 76. While the damage to the barriers is not considered severe it is recommended that repairs be made to restore the panels to their original condition (Maintenance Item M-10). A detailed list of observations made during the inspection for both the east and west railings are included in Appendix F.

However, BC MoT may elect to replace the railing entirely. It is important to note that the existing railing likely does not meet the current code requirements for a traffic barrier and it is believed that it would offer limited resistance in keeping an errant vehicle from leaving the bridge deck. With this in mind, BC MoT may wish to install a new traffic barrier along the bridge in place of the existing one (Rehabilitation Item R-15).

Figure 76: Bowed Railing Panels

4 Closing

B&T's scope of work for the Old Spences Bridge included performing a detailed inspection of the structure, evaluation of portions of the bridge and development of conceptual repairs.

This report contains observations made during B&T's 2009 inspection and makes recommendations regarding areas to focus on as part of the evaluation as well as listing items for maintenance and future inspection. Recommendations for rehabilitation items have also been provided based on the inspection findings. This report does not address the cost effectiveness of carrying out the items identified above.

B&T Report No. 1884-RPT-SPE-002-0, entitled "Load Capacity Evaluation & Rehabilitation Options - Old Spences Bridge No. 2411" summarizes the findings of the load evaluation of the bridge, makes recommendations regarding conceptual rehabilitation options, and summarizes cost estimates to restore the bridge to a range of acceptable levels of reliability.

Buckland & Taylor's 2009 Inspection of the Old Spences Bridge found that overall the bridge is in poor condition, but also identified many areas that are in very poor condition. Some of the areas in very poor condition may affect the capacity of the bridge to safely carry vehicular, pedestrian, or snow loads. Since it is not possible to establish the load carrying capacity of the bridge based on a visual inspection, a load capacity evaluation of the bridge must be carried out to determine whether it is safe to reopen the bridge to traffic and what level of traffic (i.e., load posting) can safely use the bridge.

A series of recommendations have been developed based on the inspection results and these recommendations have been broken down into:

- Evaluation Items, those items that are included in the evaluation portion of B&T's scope;
- Rehabilitation Items, those items that will be included as conceptual rehabilitation options;
- Maintenance Items, those items that are expected to be included as annual maintenance items provided that BC MoT decides to re-open the bridge; and
- Inspection Items, those items that require continued monitoring.

The Maintenance and Inspection Items have been assigned a priority rating of 1,2, or 3 indicating the work related to that item should be performed within the next 1,5 or 10 years, respectively. The maintenance and rehabilitation recommendations must be considered in conjunction with the results of the load evaluation to determine whether or not it is cost-effective to carry them out. The list of recommendations is as follows:

Evaluation Items

Item	Description
E-1	Evaluate Pier 3 for the effect of the seized bearings; refer to Section 3.4.
E-2	Evaluate the effects of localized areas of section loss in the web of the top chord; refer to Section 3.5.2.
E-3	Evaluate the effects of the hole in the top chord cover plate in Span 2; refer to Section 3.5.2.
E-4	Evaluate the effects of localized section loss in the bottom chord members in Spans 1, 2 and 5; refer to Section 3.5.3.
E-5	Evaluate the effects of localized areas of section loss in the web of the bottom chord members at the reinforcing plate locations in Spans 3 and 4; refer to Section 3.5.3.
E-6	Evaluate the effects of the localized areas of 8mm of section loss in the top flange of the bottom chord in Span 4; refer to Section 3.5.3.
E-7	Evaluate the impact of the observed corrosion on the bottom chord panel point gusset plates; refer to Section 3.5.6.
E-8	Evaluate the existing bridge deck based on the inspection observations; refer to Section 3.5.10.
E-9	Evaluate the capacity of the longitudinal stringers based on the recommended criteria; refer to Sections 3.5.11.1 and 3.5.11.2.
E-10	Evaluate the capacity of the floorbeams based on the recommended criteria; refer to Section 3.5.12.
E-11	Evaluate the effects of the impact damage to the Span 6 edge girder; refer to Section 3.6.

Rehabilitation Items:

Item	Description
R-1	Repair or replace the sliding bearings in the girder spans and the shorter truss spans; refer to Section 3.4.
R-2	Repair of replace the Pier 3 bearings; refer to Section 3.4.
R-3	Apply a surface tolerant material (Termarust or equivalent) to corroding areas to slow or stop additional corrosion; refer to Sections 3.5.1, 3.5.2, 3.5.5, 3.5.7, 3.5.11.1 and 3.5.11.2.
R-4	Recoat the entire bridge structure; refer to Sections 3.5.1, 3.5.3, 3.5.5 and 3.5.9.

Item	Description
R-5	Clean, recoat or replace the necessary bottom chord batten plates; refer to Section 3.5.3.
R-6	Replace or clean and recoat the gusset plates between the truss verticals and the bottom chord; refer to Section 3.5.4.
R-7	Clean and recoat the bottom chord lateral bracing gusset plates; refer to Section 3.5.7
R-8	Reinforce the jacking beams in Spans 1, 2, 3, 4 and 5 if required to repair bearings; refer to Section 3.5.8.
R-9	Replace sway bracing members with perforations and repair or replace as necessary, the sway bracing gusset plates with observed section loss and perforations; refer to Section 3.5.9.
R-10	Develop rehabilitation options for the concrete deck; refer to Section 3.5.10.
R-11	Repair the holes in floorbeams 4 and 8 in Span 4; refer to Section 3.5.12.
R-12	Develop options to address the gaps between the stringers and the concrete deck due to rust jacking; refer to Section 3.5.12.
R-13	Extend drain pipes in truss spans to ensure that the deck run-off drains below the bottom chord to prevent future localized corrosion and section loss; refer to Section 3.5.13.
R-14	Install shim plates beneath the stringer bearing plates in Spans 6 and 7 and install bearing stiffeners on the edge girders at their bearing points; refer to Section 3.6.
R-15	Consider installing a new traffic barrier along the bridge deck; refer to Section 3.7.

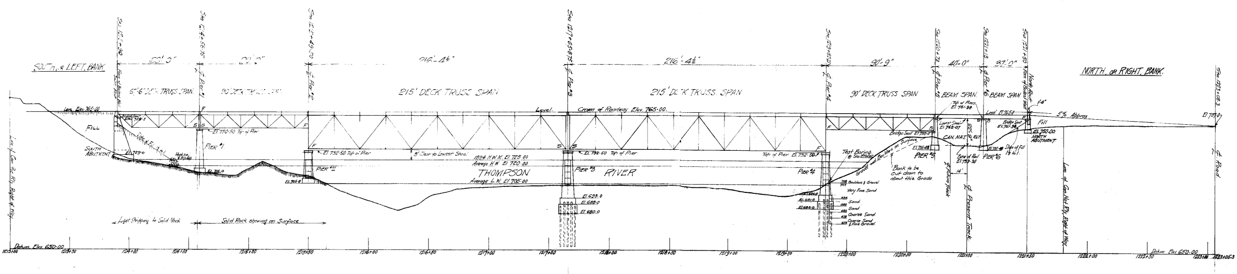
Maintenance Items:

Item	Description	Priority Rating
M-1	Reinforce the east and west sides of the North Approach roadway and fill in the void under the roadway; refer to Section 3.1.	1
M-2	Repair the two areas of distressed asphalt on the South Approach; refer to Section 3.1.	1
M-3	Replace the no-post barrier on the south approach that is supporting the fill with a well anchored support; refer to Section 3.1.	1
M-4	Paint over the graffiti on the face of both abutments; refer to Section 3.2.	2
M-5	Repair the two concrete spalls on the north face of Pier 5 and fill the horizontal cracks with a product similar to Sikaflex 2C NS; refer to Section 3.3.5.	2
M-6	Paint over the graffiti on the face of Pier 5; refer to Section 3.3.5.	2
M-7	Paint over the graffiti on the face of Pier 6; refer to Section 3.3.6.	2
M-8	Install a fully tensioned bolt in the empty rivet hole at member L6-U7 in Span 4; refer to Section 3.5.5.	2
M-9	Pressure wash the areas of cracking in the bridge deck and apply a silane sealer; refer to Section 3.5.10.	1
M-10	Repair the damage to the railings on both sides of the deck; refer to Section 3.7.	2

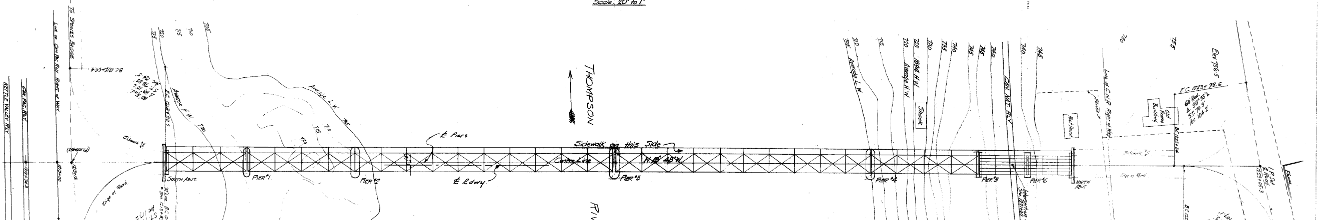
Inspection Items:

Item	Description	Priority Rating
I-1	Monitor the erosion on the South Embankment under Span 1; refer to Section 3.1.	Ongoing
I-2	Monitor the condition of the concrete at the cracks in the North Abutment; refer to Section 3.2.	Ongoing
I-3	Monitor the condition of the Pier 1 concrete; refer to Section 3.3.1.	Ongoing
I-4	Monitor the condition of the Pier 2 concrete; refer to Section 3.3.2.	Ongoing
I-5	Monitor the condition of the Pier 3 concrete; refer to Section 3.3.3.	Ongoing
I-6	Monitor the condition of the Pier 4 concrete; refer to Section 3.3.4.	Ongoing
I-7	Monitor the condition of all cracks in the Pier 6 concrete; refer to Section 3.3.6.	Ongoing
I-8	Monitor the areas of section loss in the batten plates at the end of the diagonals; refer to Section 3.5.5.	Ongoing
I-9	Monitor the condition of the bottom chord lateral bracing gusset plates; refer to Section 3.5.7.	Ongoing
I-10	Monitor the performance of the mastic material in the deck joints; refer to Section 3.5.10.	Ongoing

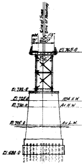
Appendix A
General Arrangement Drawing



PROFILE LOOKING DOWNSTREAM.
Scale. 20' to 1"



PLAN
Scale. 20' to 1"



SECTIONAL ELEVATION
AT PIER #3

INDEX TO DRAWINGS

Formerly Dwg. No. 757-1

YALE DISTRICT.

PROPOSED BRIDGE
AT SPENCES BRIDGE,
— OVER THE THOMPSON RIVER. —

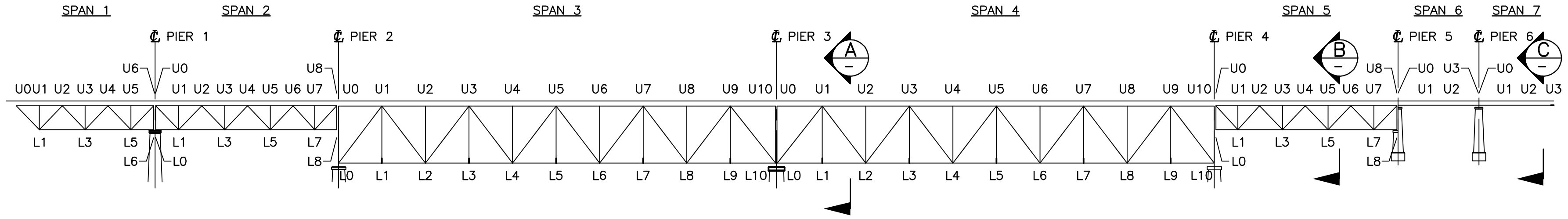
LAY-OUT PLAN.

Scale - as noted

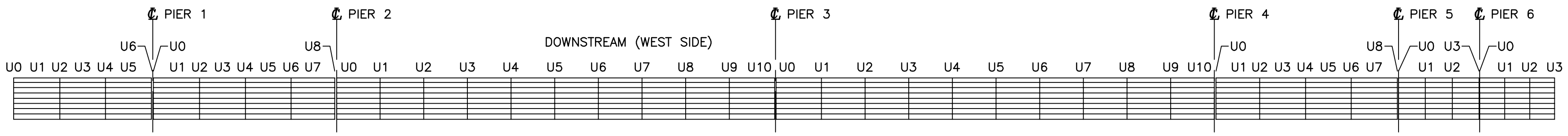
GOVERNMENT OF BRITISH COLUMBIA,
DEPARTMENT OF PUBLIC WORKS,
VICTORIA, B. C.

Drawing No. 2611 Sheet No. 1
Made by G.M.D. Date May 1929
Traced by .... Date ....
Checked by J.T.H. Date 1929
Examined and Recommended for Approval
[signature]
APPROVED. [signature]
Deputy Minister and Public Works Engineer.
Assistant Public Works Eng'r.

Appendix B
Numbering Convention



ELEVATION
SCALE 1:500



STRINGER AND FLOOR BEAM – PLAN
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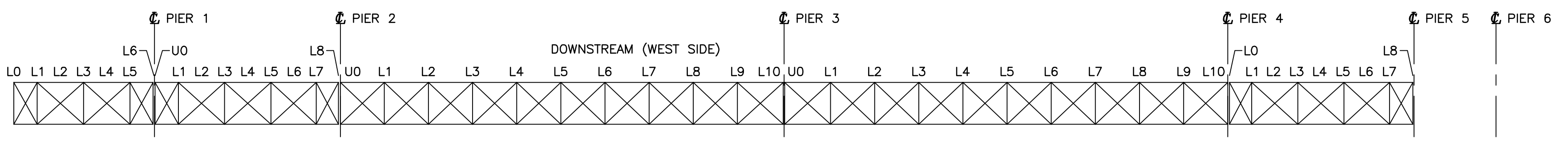

BOTTOM CHORD AND BRACING – PLAN
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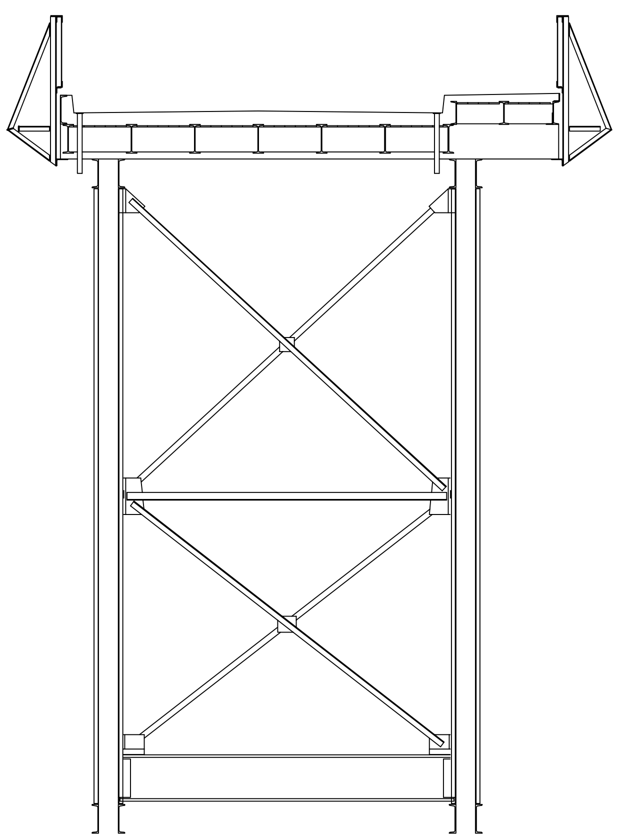
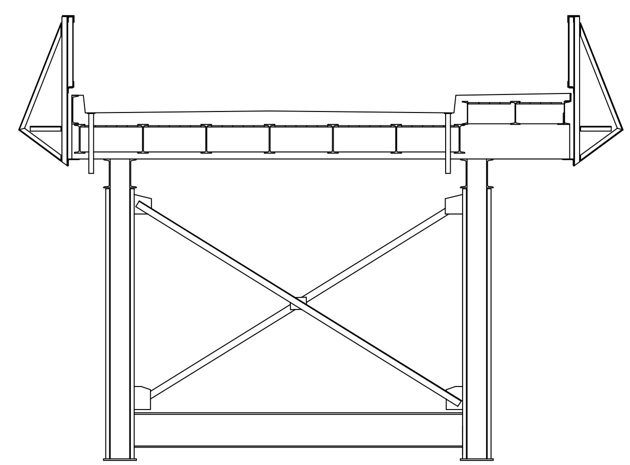


SECTION A
SCALE 1:50



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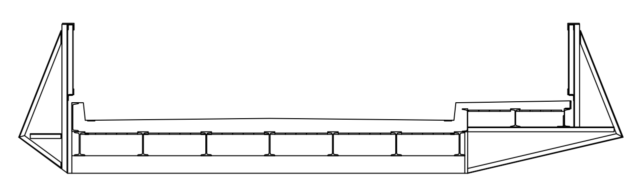


SECTION C
SCALE 1:50

Consultant Logo
BUCKLAND & TAYLOR LTD. Bridge Engineering

Rev	Date	Description	Init
PA	09/12/18	FIRST ISSUE	AG

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE NUMBERING CONVENTION

PREPARED UNDER THE DIRECTION OF
ENGINEER OF RECORD
DATE
SEAL

DESIGNED MS DATE 2009-12-18
CHECKED AG DATE 2009-12-18
DRAWN PBR DATE 2009-12-18
SCALE AS SHOWN
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-N01 PA

File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-N01.dwg Save Date: Dec 18, 2009 1:34 PM Saved By: PREES

Appendix C
Bridge Management Information System Condition Inspection Sheets

BRITISH COLUMBIA | Ministry of Transportation and Infrastructure

BRIDGE CONDITION INSPECTION

Inspection Type
Routine ☐
Detailed ☒

Structure Number: 2411
Structure Name: Old Spences Bridge
Inspection Date (yyyy/mm/dd): 2009/10/27

COMPONENT | **PERCENT CONDITION RATING** (Enter % in each condition. See BMIS User Manual 15.2.2) | **INSPECTION NOTES BY COMPONENT** (All poor or very poor conditions should be explained with notes and documented by photos. Label explanation(s) with component)

#	Component	E	G	F	P	V	X	N	Inspection Notes
	CHANNEL								
1	Debris Risk	100							See attached sheet for notes
2	Bank/Bed	100							
3	Dolphins/Fenders							100	
	SUBSTRUCTURE								
4	Foundation	100							
5	Abutments		100						
6	Wing/Retaining		100						
7	Footings/Piling						100		
8	Pier			70	30				
9	Bearings			30	40	30			
10	Caps		20	60	20				
11	Corbels							100	
	SUPERSTRUCTU								
12	Floor		50	40	8	2			
13	Stringers		60	25	10	5			
14	Girders		80	15	5				
15	Portals							100	
16	Bracing/Diaphragms		70	10	15	5			
17	Truss Chords/Arch		80	10	9	1			
18	Arch Ties							100	
19	Truss Diagonals		80	15	5				
20	Truss Rods/Verticals		80	10	5	5			
21	Cables							100	
22	Panels							100	
23	Pins/Bolts/Rivets		93	5	2				
24	Camber/Sag		100						
25	Live Load Vibration						100		
26	Coating (structure)			20	55	25			
	DECK								
27	Sub Deck/Cross Ties			80	15	5			
28	Wearing Surface		75	25					
29	Deck Joints				95	5			
30	Curbs/Wheelguards			85	5	10			
31	Sidewalk(s)			80	20				
32	Railings/Parapets			90	10				
33	Median Barrier							100	
34	Drains/Pipes			100					
35	Coating (Railings)				30	70			
	APPROACHES								
36	Signing/Lighting						100		
37	Roadway			70	30				
38	Roadway Flares						100		

General Inspection Notes:
See attached sheet

Urgency Rating Notes:
Must do load rating given significant corrosion see attached

Utility Concern Notes (Contact Utility Owner):
Insulation on water pipe above various piers is missing.

Condition Codes

E	Excellent	V	Very Poor
G	Good	X	Not
F	Fair	N	Not
P	Poor		

For Condition Guidelines see BMIS User Manual 15.2.2

Urgency Rating
5

For definition see User Manual 15.2.8 "4" and "5" rating be explained.

EER / TD (General & Structural) RR (Coatings)
Inspector (please type or print)

Signature

Structure Number 2411

Posted Weight Restriction *(print actual message on sign(s))*

Other Posted Hazard Warning Signs

Bridge is currently closed to vehicle traffic

Drainage Area Description *(water level fluctuation, logging debris, etc.)*

The bridge crosses the Thompson River where it is relatively wide. However, it can experience water level fluctuation and some minor debris during the spring run-off.

Rehab Work Notes

As the inspection of Old Spences Bridge resulted in defining Evaluation Items, Rehabilitation Items, Maintenance Item and Inspection Items, please refer to the Buckland & Taylor Ltd. Report 1884-RPT-SPE-001 (Section 4: Closing) for a summary of each recommendation type and specific recommendations.

Maintenance Work Notes

As the inspection of Old Spences Bridge resulted in defining Evaluation Items, Rehabilitation Items, Maintenance Item and Inspection Items, please refer to the Buckland & Taylor Ltd. Report 1884-RPT-SPE-001 (Section 4: Closing) for a summary of each recommendation type and specific recommendations.

Urgency Rating Notes

Some of the areas in poor or very poor condition may affect the capacity of the bridge to safely carry vehicular, pedestrian, or snow loads.

A load capacity evaluation of the bridge must be carried out to determine whether it is safe to reopen the bridge to traffic and what level of traffic (i.e., load posting) can safely use the bridge.

Inspection Notes by Component

Comments listed below are general. For more details please refer to the Buckland & Taylor Ltd. Inspection report 1884-RPT-SPE-001.

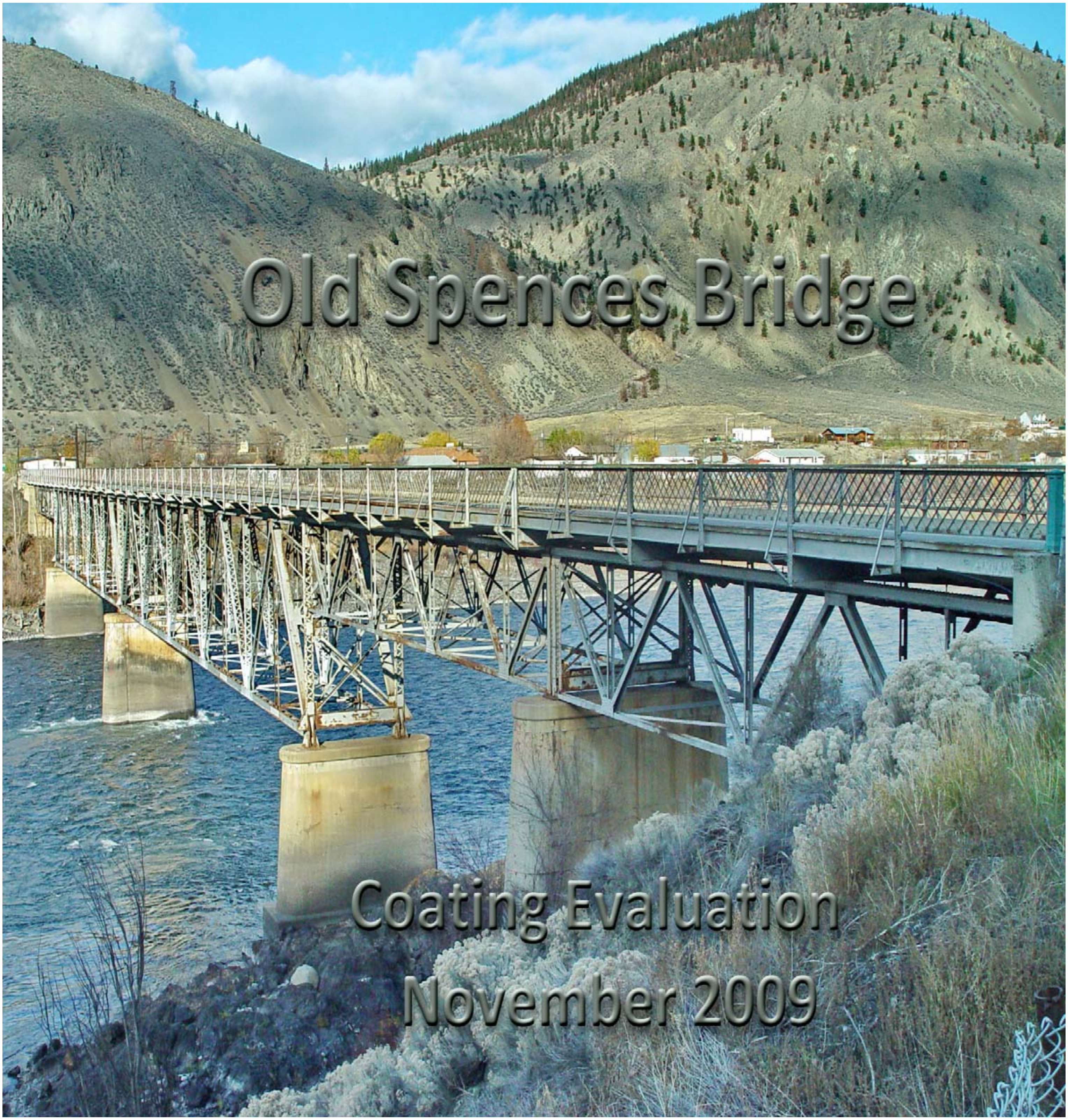
Component		Note
1	Debris Risk	Minimal deposits of debris in the river, on the banks. No deposits of debris on the structure.
2	Bank/Bed	No observed scour or build up around piers, abutments or banks.
4	Foundation Movement	Most piers are founded on bedrock, no evidence of movement observed.
5	Abutments	Normal wear and deterioration, no structural repair or maintenance required. Medium cracking observed on the north wall with minor efflorescence.
6	Wing/Retaining Wall	Normal wear and deterioration, no structural repair or maintenance required. Light/medium cracking observed on the north abutment east face wing wall.
8	Pier Columns/Walls/Cribs	Numerous cracks observed in all faces ranging from hairline to wide (3-4mm). Locations of small to large areas of degraded concrete noted on most faces of unreinforced piers. Delaminations and spalls observed on piers with reinforcing steel.

Component		Note
9	Bearings	Many locations of severe pack rust and moderate to heavy corrosion of the exposed steel surfaces, likely limiting the amount of movement that can be accommodated by the bearings. No obvious signs of longitudinal movement on any of the sliding bearings. Many of the anchor bolts were observed to be bent, one was sheared off (SE bearing Span 4) and another 3 exhibited an area of reduced cross section (SW bolt at NW bearing of Span 3 and east bolts at SE bearing of Span 4). Wear patterns indicate, where designed to allow rotation, the bearings still permit this.
10	Caps	Cracking in most faces ranging from hairline to wide (6-7mm). Locations of small to large delaminations on most of the east and west faces.
12	Floor Beams/Transoms	Generally, the coating system as failed and light to moderate surface corrosion exists. Some locations were observed with heavy corrosion and more than 15% section loss. Two locations with complete section loss were observed (see B&T Inspection Report).
13	Stringers	Within the middle 60% of the span length the stringers generally exhibit normal wear and deterioration not requiring repair. Nearer to the deck joints, some stringers exhibited light corrosion with no measurable section loss. Others exhibited moderate corrosion with some section loss. Also, near the deck joints the coating system has failed on all stringers. Previous repairs observed, some repairs called for on the stringers but not yet installed. Some sections have heavy corrosion with more than 15% section loss. Select locations (see B&T Inspection Report) were observed with full section loss.
14	Girders	Localized areas of coating failure. Rust jacking was observed to be adversely affecting the load path in one location: Span 6 girder webs appear to have buckled slightly at bearing point at Pier 6. Span 6, west girder: flange bent upwards assumed to have been caused by impact from train cargo.

Component		Note
16	Bracing/Diaphragms	Some locations of coating failure and surface corrosion, generally in good condition. Select locations of section loss (primarily beneath deck joints) along the members. Some gusset plates are in poor condition with perforations. Jacking beams are generally in fair condition with some localized perforations.
17	Truss Chords/Arch Ribs	Generally the chords are in good condition away from the deck joints. Closer to the deck joints and the deck drains, some localized areas of light to moderate corrosion were observed. On the top chord, one perforation of a top cover plate was observed. At some panel point locations on the bottom chord, moderate section loss was observed on the top flange and web at the perimeter of the gusset plates.
19	Truss Diagonals	Generally the diagonals are in good condition away from the deck joints. Closer to the deck joints and the deck drains, some localized areas of light to moderate corrosion were observed. At some panel point locations at the bottom chord, moderate section loss was observed on the gusset plates at the perimeter of the bottom chord and select locations.
20	Truss Rods/Verticals	Generally the verticals are in good condition along their length, some localized areas of light corrosion were observed. At some panel point locations for the bottom chord, moderate section loss was observed on the gusset plates at the perimeter of the bottom chord and select locations. Perforations were also observed at the gussets connecting the vertical member to the bottom chord at odd numbered panel point locations. There is no immediate structural concern for these vertical member gusset plates, even though some have significant perforations, since the gusset plates with significant section loss/perforations all connect zero force vertical truss members to the bottom chord of the truss (i.e., they carry very little load).
23	Pins/Bolts/Rivets	Light corrosion was observed on most rivets. At the top chord panel points, some rivets were observed to have moderate corrosion product build up. No pitting or appreciable section loss was observed.

Component		Note
24	Camber/Sag	Excessive sag or camber was not observed.
25	Live Load Vibration	None observed since bridge closed to vehicle traffic.
26	Coating (Structure)	Generally, the structure's coating system is in poor condition with advanced corrosion all across the bridge. For additional comments, refer to Appendix D of B&T Inspection Report.
27	Sub Deck/Cross Ties	Honeycombing, cracking, exposed reinforcement (underside) and locations of delaminations were observed.
28	Wearing Surface	Some cracking was observed at the corners of the concrete deck panels. Otherwise, in good condition.
29	Deck Joints	Generally the joints do not appear to restrict the flow of water and debris from the steel and concrete below deck. Water drains through at the joint ends directly onto the floorbeams and then runs along the bottom flange.
30	Curbs/Wheelguards	Curbs are in fair condition with locations of spalling and cracking along their length. Some locations were observed to have previous repairs which do not appear to be performing well.
31	Sidewalk	Numerous delaminations along its length.
32	Railings/Parapets	Damage observed in multiple locations, light surface corrosion.
34	Drains/Pipes	Drain hardware in good condition (note: drains direct water onto the truss members).
35	Coatings (Railings)	Approximately 50% of the coating was missing. For additional comments, refer to Appendix D of B&T Inspection Report.
37	Roadway Approaches	On the north approach, soil immediately behind the abutment was observed to be sloughing away creating a void beneath the roadway surface. Cracking was noted in the asphalt on both approaches.

Appendix D
Report on the Existing Coating on the Old Spences Bridge



Trans Canada Coatings Consultants Ltd.
1059 Hampshire Rd., Victoria, BC V8S 4S8
transcoat@shaw.ca, 250 598-1030

Coating Assessment Old Spences Bridge
November 2009

General Comments:

1. Overview: Old Spences Bridge

Old Spences Bridge was built in 1931 at Spences Bridge to carry Hwy 1 across the Thompson River. It consists of 7 spans, 5 underdeck truss spans and 2 stringer spans. The bridge was replaced by a structure further downstream, but the older structure was retained to carry local traffic at the Town of Spences Bridge across the river without having to detour to the new structure. We have no firm data, but it appears the paint may be original, since there is millscale underneath and we did not see any abrasive blasted steel. Generally the structure's coating system is in poor condition with advanced corrosion all across the bridge. Currently the bridge is closed to vehicle traffic until further evaluated.

2. The Existing Coating System

The existing coating system appears to be a three coat red lead / alkyd / alkyd system although this was confirmed at only one point.

The existing paint is in poor condition and has not been protecting the steel for some time. There is peeling scattered over the structure which indicates additional risk for overcoating

Adhesion values were estimated at < 2B (ASTM D3359 Method B) where tested. This contrasts with adhesion data taken in September 1996 which indicated roughly 4B. Delamination occurred between the primer and the heavy mill scale at the one location we investigated. The coating is brittle and chipped which is another indicator of the poor condition of the existing paint.

3. Inspection

The inspection was done on November 13, 2009. Air and steel temperatures were between 6 and 11 degrees Celsius. Weather varied from dry and cloudy to driving sleet.

Temperatures were too low to perform the normal ASTM D3359 adhesion test, so a new cross hatch cut was done to in an attempt to compare with one done in the Sept. 1996 inspection.

The inspection was curtailed when bad weather moved in later in the afternoon. Approximately 90% of the data was collected at that point, so further inspection was judged unnecessary, particularly since the report deadline was only 7 days hence.

For the inspection, the bridge was divided into individual spans and further subdivided into components of the individual spans so as to reflect the various microclimates experienced by the bridge coatings.

The components inspected from top to bottom were:

- Fascia – Generally the outsides of the painted members exposed to sunlight. For this inspection it includes the outsides of the longitudinal chords below deck as well as most of the underdeck truss work.
- Protected area – Generally the area under the bridge protected from the elements. This area does not include areas under joints or above bearings as these areas are looked at separately.
- Cross Bracing – this area underdeck is looked at separately as in many cases, the cross bracing seems to age faster than the general protected area.
- Deck Joints – This includes the transverse members under the deck expansion joints and is inspected separately as most bridges have some joint leakage with resulting coating degradation.
- Bearings – The bearings area includes the bearings and the area above up to the transverse members which are typically part of the deck joints.

Once the component level inspections are done, the results can be rolled up to span and bridge level to get an overall picture.

4. Scope of Work

Most of the inspection was done visually, using ASTM D610, and SSPC-VIS 2: Standard Method of Evaluating Degree of Rusting on Painted Steel Surfaces visual standards to evaluate the amount of rusting.

Dry film thicknesses (DFT) taken with a Positector 6000. Sixty readings were taken over the steel at the south end of Span 1. Readings are in mils (thousandths of an inch, 1mill = 25.4μm).

A Tooke reading was done on the truss steel at the south end of Span 1. The Tooke readings reveal how many coats of paint and how thick each coat is. Readings are in mils (thousandths of an inch).

Pictures were taken of significant or typical areas as well as overall. A few pictures are included in the report, but all are available on a CD if needed.

The bridge was inspected from the ground without benefit of lifts or ladders since it was a fairly simple structure and most parts were visible. We did not directly access the underside of the structure except at the south end; therefore no dry film thicknesses (DFT's) or Tooke readings were done on the middle span underdeck. The bearing areas were judged from pictures except at the abutment bearings.

5. Observations

The coating on the structure appears to be a three coat alkyd with red lead primer over much of the structure. It appears to be in poor condition and not to be protecting the steel. The substrate in all sample areas appeared to be millscale. The paint in general is in a "normal" surface condition, that is, with normal amounts of chalking, dirt, etc., however it is badly failed over much of the surface and quite brittle. The adhesion has gone down considerably since a previous inspection carried out in 1996.

Presence of Red Lead

This Structure contains red lead and appropriate mitigative measures will have to be taken to prevent the spread of lead into the immediate environment should coating work go ahead.

Adhesion

Temperatures were not adequate to do the cross hatch adhesion testing. Non-peeling areas were estimated to be in the sub 2B area. This is poor adhesion. We were not able to access the coating except at the south abutment, span 1.

Figure 1 Attempted Adhesion Comparison. On the left is the original crosshatch done in Sept. 1996 after the tape pull. On the right, is the current crosshatch, but without the tape pull. Paint has spontaneously delaminated from the scribing of the lines. Even without the tape pull it is much worse than the 1996 test.

Peeling

Peeling was in evidence over the structure as evidenced by the two pictures above and the picture following. There was significant peeling in the protected area underdeck scattered over the bridge and on the fascias.

Figure 2 Span 1 First floor beam. Peeling and advanced corrosion.

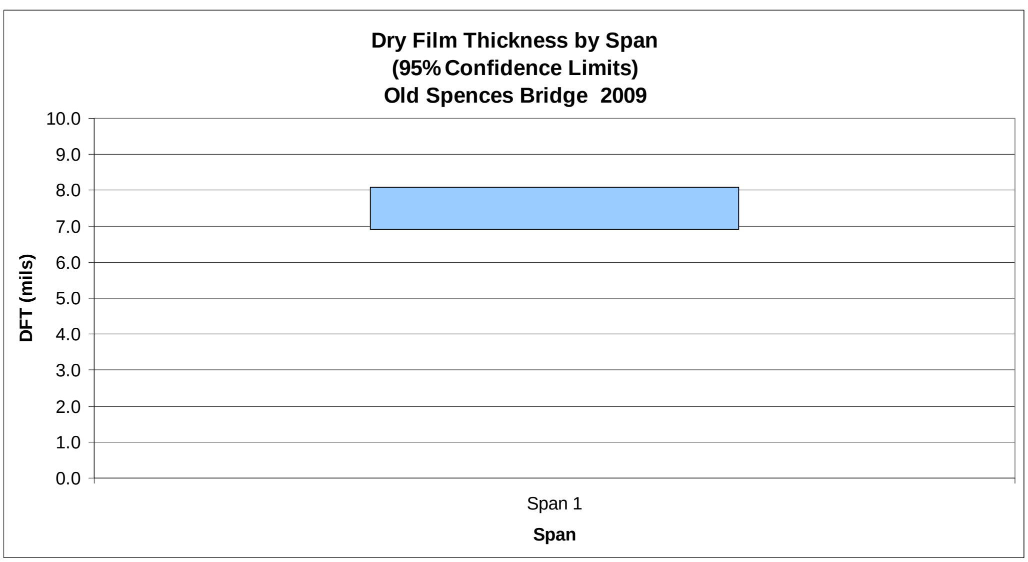
Figure 3 Span 1 North side looking toward the span 1-span 2 bearing area.

Typically, where there is peeling or other coating failure there is considerable corrosion. Other coating defects were noted on Span 1

Figure 4 Alligator cracking was observed on the bottom of a deck stringer on Span 1. Some was also noted on the stringer webs in the protected area on Span 5. This appeared to be in fairly isolated areas, and was possibly an application error or insufficient cleaning before painting.

Coating Film Thickness

The coating film thickness was taken on span 1 only. A total of 60 readings were taken at the abutment area. The 95% confidence limits for the readings on span 1 were between 6.9 and 8.2 mils, a little thicker than normal for alkyd coatings. The coating was fairly rough with considerable coarse particles painted in (sand?)



Railings

Coatings on the railings are poor. The average ASTM D-610 either side varied from 1.4 to 2 out of ten. About 30 to 50% of the coating was missing. As well, considerable damage was noted from impacts.

Figure 5 Section of the Handrail at the Span1, Span 2 Joint. The post and rail have experienced impact damage. Sections are missing and bent. Paint failure here is a little less than on the handrails as a whole. Many of the bottom gussets are corroded through.

Protected Area

The protected area up under the deck is at an average condition of 3.8 which corresponds to an overall failure of about 10% of the surface. The steel in areas that are failed are showing considerable corrosion.

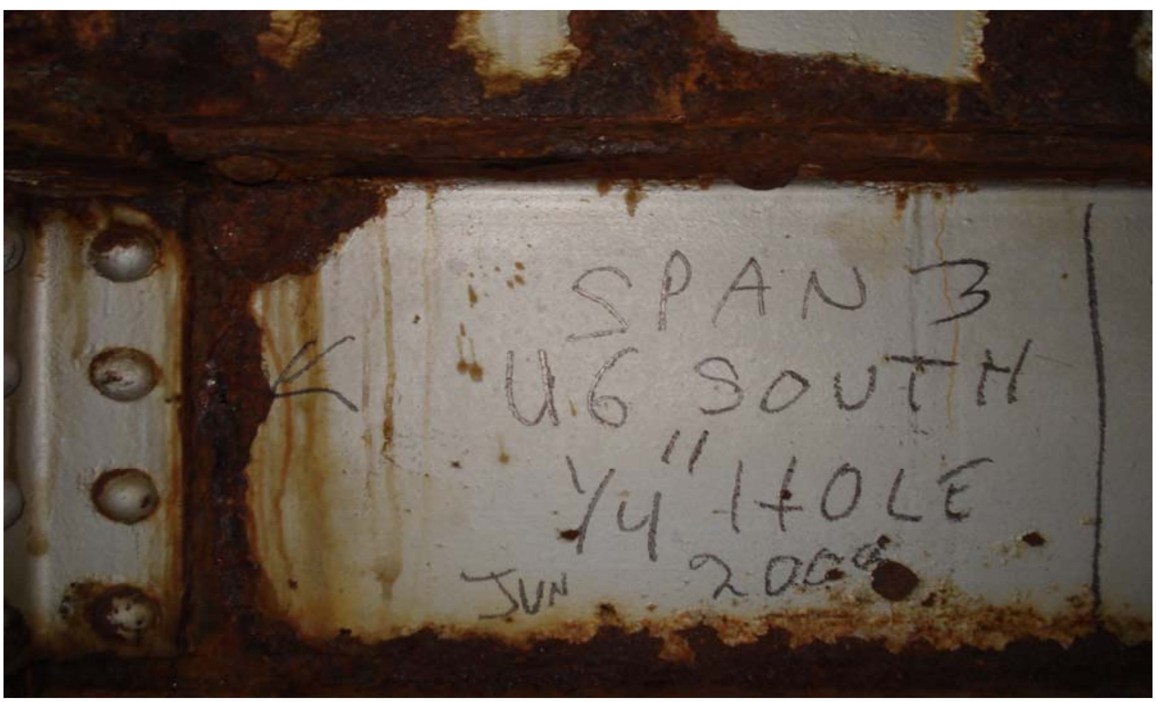


Figure 6 The protected area under Span 3. This is a little worse than average, but by no means unique. The steel is showing heavy corrosion including section loss and pack rusting. Photo courtesy of Buckland and Taylor Ltd.


Figure 7 View of Span 1 Protected Area Looking North. This is fairly typical of the whole Bridge.


Figure 8 Span 5 Floor beam. This shows fairly heavy corrosion where the coating has failed. Note the checking pattern on the bottom of the stringer flanges. Although at first glance the paint appears reasonable, closer inspection reveals almost complete coating failure

Overall, the coating shows variations in the scores from 3.2 (20% failure) to 4.5(5% failure) when the data is expressed as a 95% confidence limit.

Fascias

Fascias on this structure included most of the trusswork under the bridge as it is exposed to the sun and weather. The coating scored 3.9 the West fascia and 4.1 the East fascia (about 10% failure overall). Failure was accelerated on the bottom chords which seemed to correlate with the deck drains which terminate just below the top longitudinal members on the outside of the bridge.

Figure 9 Span 2 West Side. Note coating failure and corrosion on the ends of the floor beams, on diagonals, and on the lower chord.

Figure 10 Span 2 East Side. Note the periodic nature of the failure on the bottom chords which correlates with the deck drains. Note also the deck joints where extra coating failure and corrosion have occurred.

Figure 11 Span 6 East Fascia. This is typical of the two stringer spans. Most of the failure is on the bottom chord.

Deck Joints

Expansion joints were not looked at separately. For this structure they were included in the fascias as the two areas overlapped and the condition of the paint precluded zone painting work on either separately.

Figure 12 Span 1/2 Joint and Bearing. The corrosion here has perforated the lower transverse beam.

Bearings

The bearing condition score varied from 3 (17% failure) to 5.4 (2 percent failure). These scores are at the clean to bare metal and recoat stage, although on some bearings there might be an option to defer maintenance if the associated corrosion is not too advanced.

Figure 13 Span 2 North Bearings. This is fairly typical of the truss bearings.

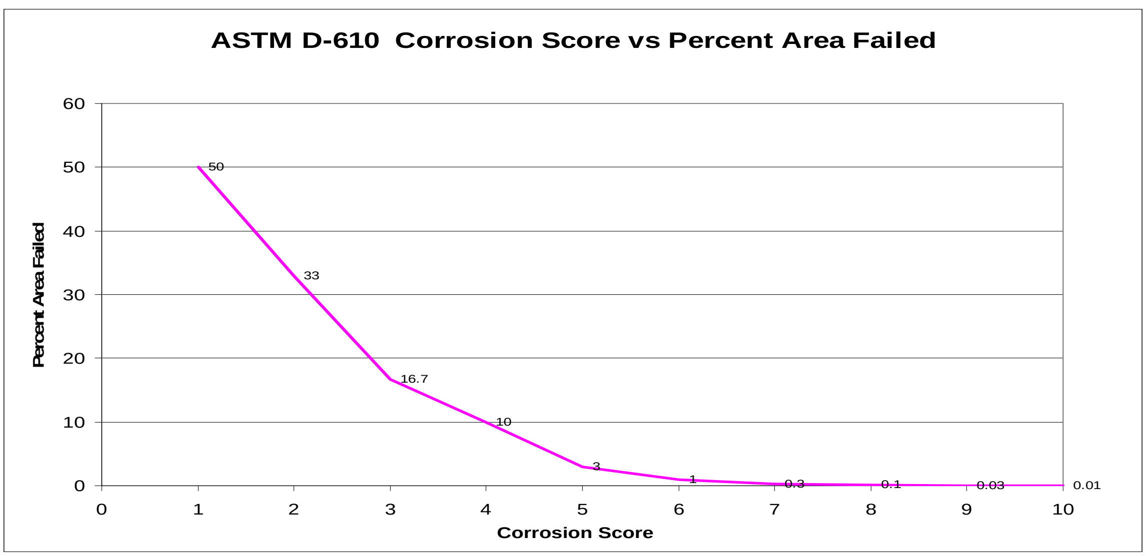
6. Data

Data were taken on an inspection visit on November 13, 2009. The tests were done at a component level for each span. That is for each span, areas such as bearings, splash zone, above splash zone, protected area, fascias etc. were judged individually as to their coating failure, adhesion, dry film thickness and so on.

The component scores here reflect the amount of area of paint failure as per ASTM D 610. Point scores are assigned as follows

ASTM D-610 Table of Score vs. Area Failed

Score	Percent Paint Failure	Equivalent to 1 Part in:
10	<0.01	10,000
9	<0.03	3333
8	<0.1	1000
7	<0.3	333
6	<1	100
5	<3	30
4	<10	10
3	<16.7	6
2	<33	3
1	<50	2

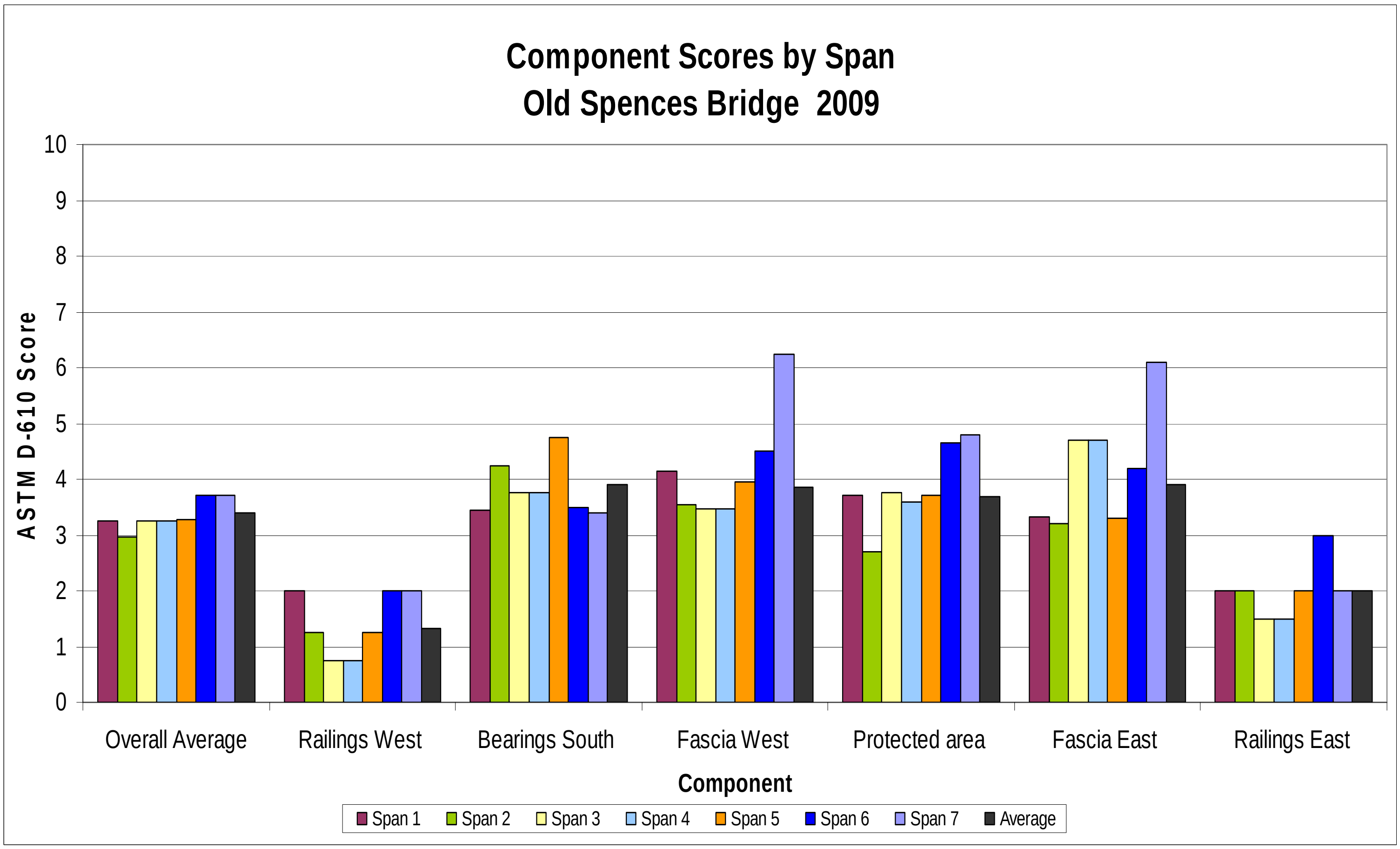


When evaluating the D-610 score, the individual components are divided into percentage areas in the assigned categories. The ultimate score for a component is calculated as the weighted average (percentage times the category number).

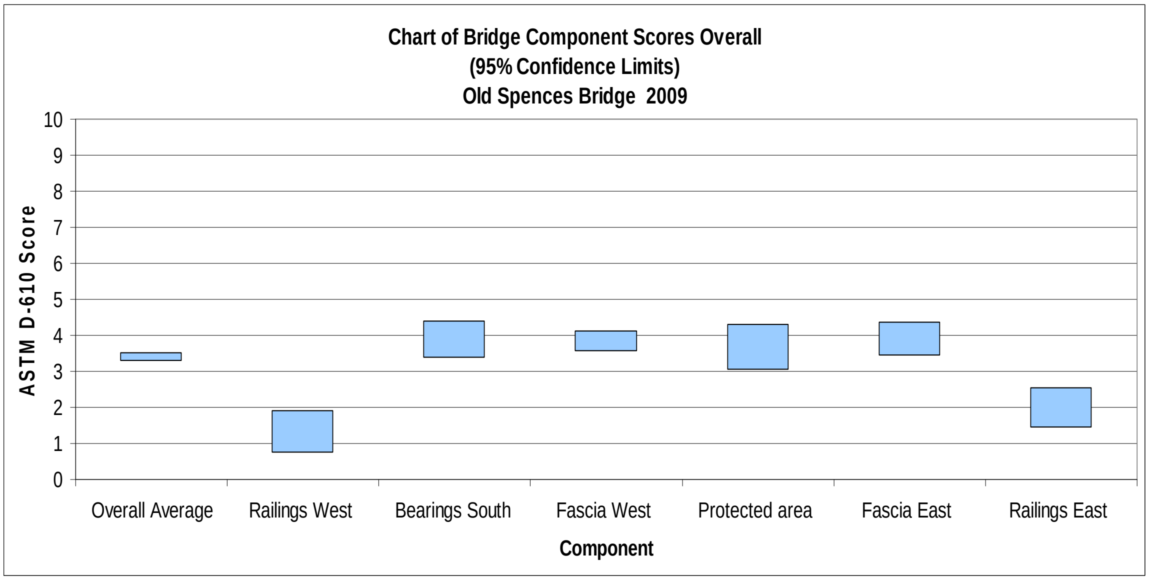
Corrosion Categories per ASTM D-610											
<.01	<.03	<0.1	<0.3	<1	<3	<10	<16.7	<33	<50	>50	% Fail
10	9	8	7	6	5	4	3	2	1	0	Avg.
		10	20		25		15		30		4.2

The table above is an excerpt from the D-610 data taken. In general, scores above 7 relate to cosmetic changes and do not affect the steel much unless the failure is localized. Below category **6**, corrosion is occurring. Data taken by the British Columbia Bridge Coating Rating System in 1996, and 2001 indicate that once failure is started, the degradation proceeds at a rate of one corrosion category every 5-10 years depending on the bridges location and the microclimate experienced by the coating.

The data presented below deals mainly with the corrosion as measured by ASTM D-610.

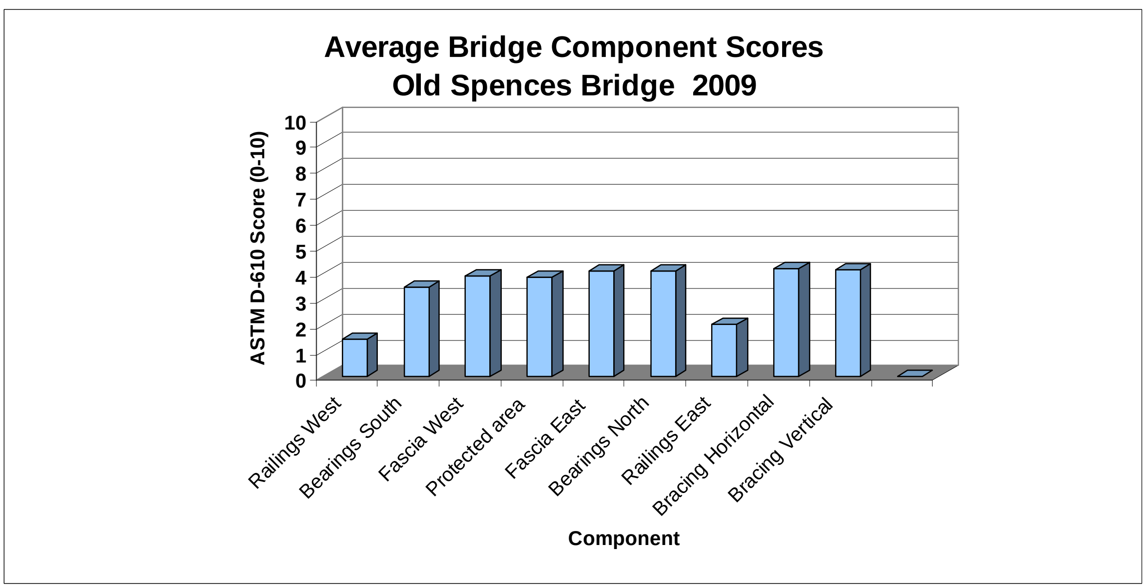


The graph above presents the average ASTM D 610 Corrosion scores by component for each of the spans. The data show that on average there is not much difference span to span implying that the coating is failing relatively uniformly over the structure. The bridge overall scored 3.4 out of 10 (about 10 to 15% coating failure). The average scores, for inspected components except the railings were in the 4 out of 10 range or about 10% failure overall.



This graph presents the 95% confidence limit of each component averaged over all 7 spans collectively. This presentation gives information on how much spread there is in the data for a given component. Overall the score was about 3.4 for the whole bridge. This indicates very little equity still present in most of the coating.

This bridge was inspected under the Bridge Coating Rating Survey in September 1996. At that time the overall score was 6. This represents a decline from about 1% failure overall then to about 10 to 15% now.



This graph presents the same data as the graph immediately above, but as an average score for each component over all seven spans.

7. Discussion:

The coating on Old Spences Bridge is badly failed and is not providing significant protection for the steel. The corrosion on the structure is widespread, with numerous instances of pack rust, scaling and section loss including perforation of steel members. Further coating remediation would have to be carried out in conjunction with steel remediation/replacement to be fully effective. It would also depend on the type of service required (pedestrian or vehicular traffic) and the required lifetime for the remediation. There are not many coating options because the coating is at the end of its useful life and is too deteriorated to overcoat. To gain useful life for the bridge the corrosion must be slowed or stopped.

8. Costing Data

The following graphs outline two alternative coating rehabilitation scenarios. Costing is added so that the overall lifetime cost may be compared as well as other benefits such as appearance. It is difficult to come up with absolute costs as there are very few comparable structures where coating rehabilitation has been done. We do feel however, that the relative costs for each scenario given below are reasonably accurate and sufficient to judge the most economical treatment over the structure's lifetime. The costs presented are for coating rehabilitation only. There are likely many areas where the steel will need to be rehabilitated or replaced.

Costing data is presented as both present day dollars and net present value dollars at 4%. Should a different discount rate be desired, it can be easily calculated as well.

9. Recommendations:

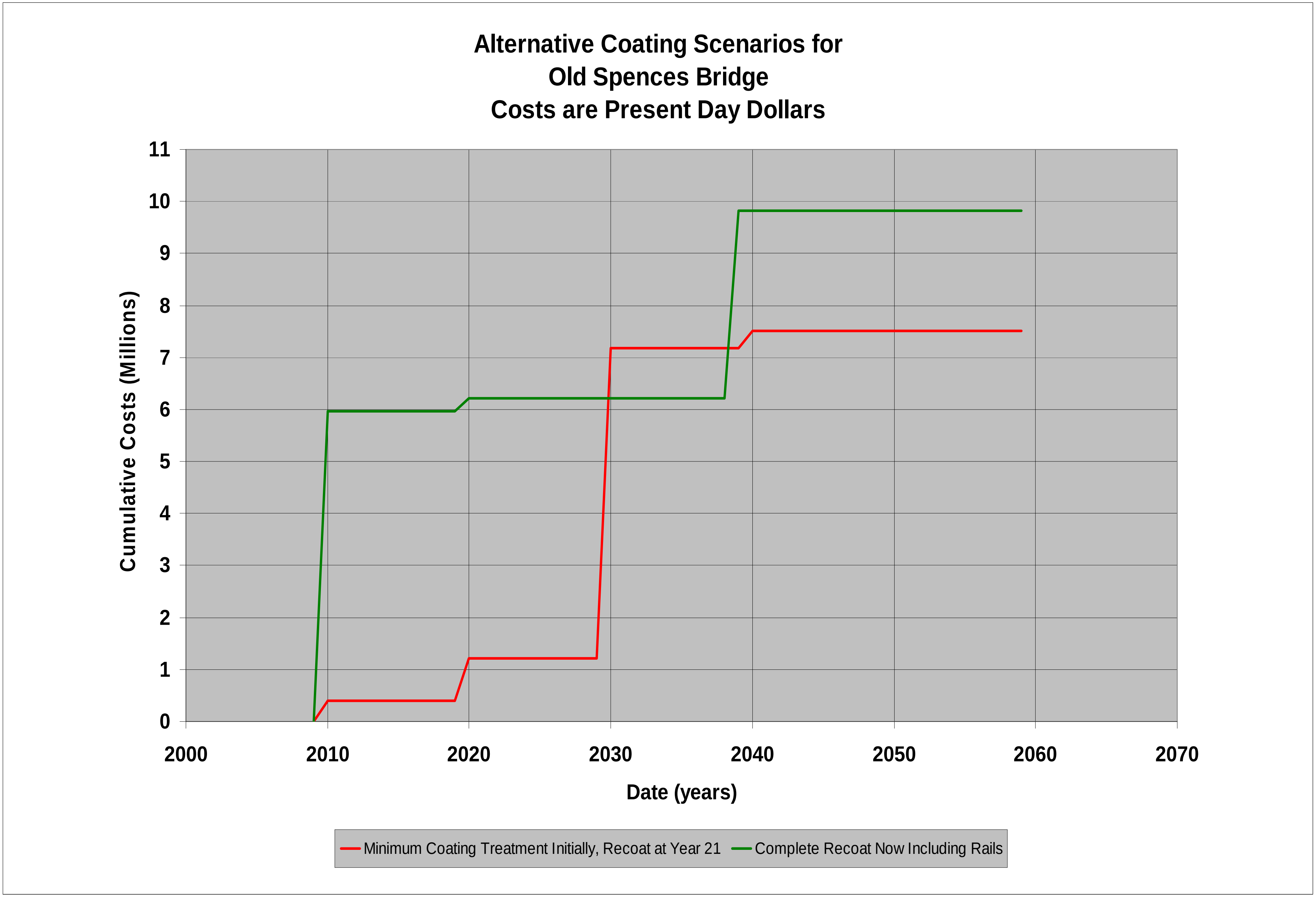
Other than doing nothing and allowing the bridge to be decommissioned in the near term; we can see only two options that would stop or slow the corrosion and gain useful life for the bridge:

The first is to immediately touchup the corroding areas with a simple, easy to use, surface tolerant material such as Termarust to drastically slow or stop additional corrosion. This would likely be done with some steel rehabilitation as well. This is the **RED** scenario in the graphs and tables below. It would likely concentrate on perhaps 25% of the surface that is in the worst condition or,

The second option is to immediately recoat the structure by stripping the bridge to bare metal, and then coating it with a three coat zinc/epoxy/urethane system (or equivalent). This means spending capital now, but results in a superior system, that is aesthetically much more pleasing. This is the **GREEN** scenario in the graphs and tables below.

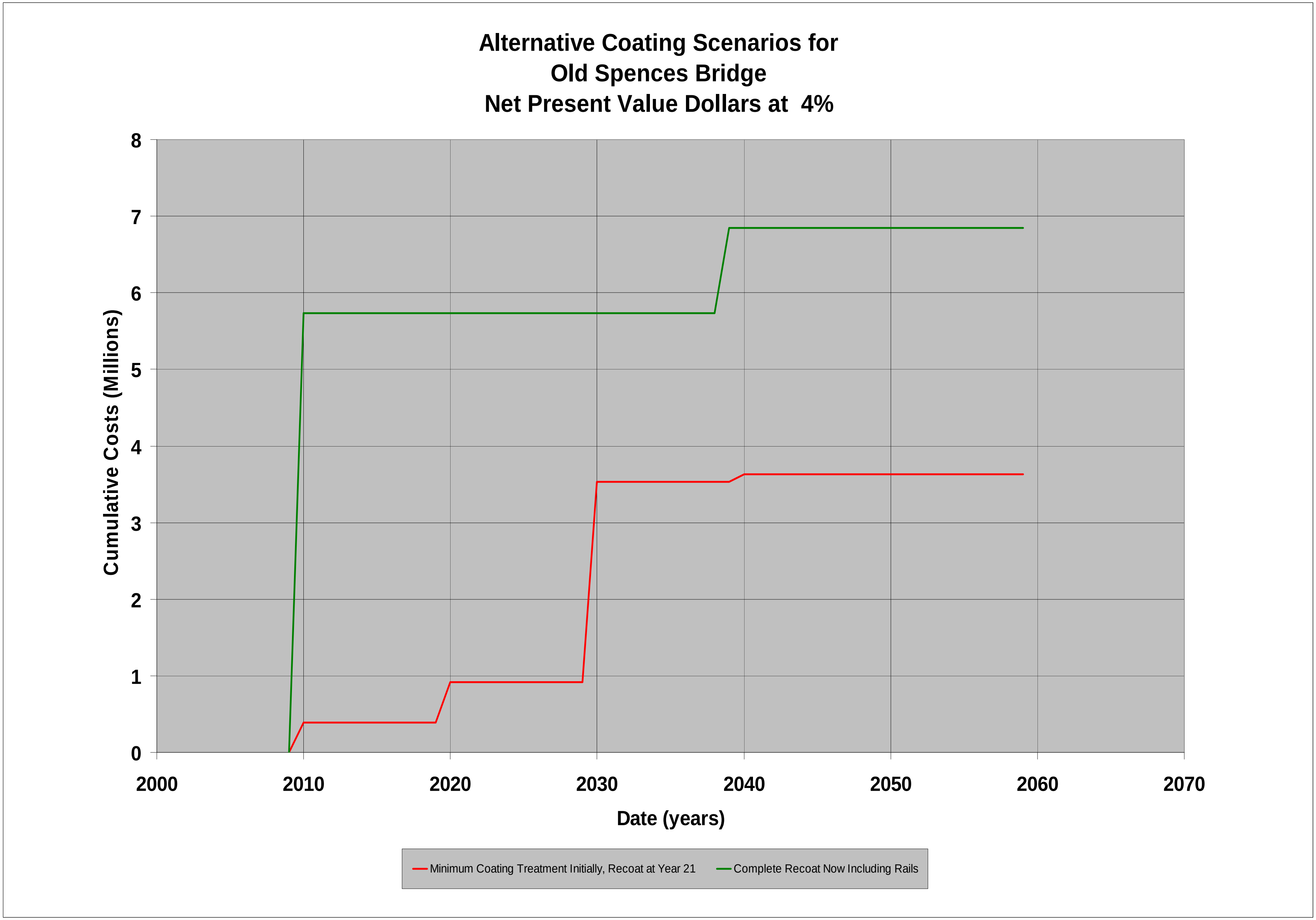
In either scenario we would recommend that the railings be hot dip galvanized or replaced with new hot dip galvanized railings to minimize their cost over the remaining lifetime of the bridge.

Appendix 1
Old Spences Bridge
Costing Scenarios


Alternative Coating Scenarios for
Old Spences Bridge
Costs are Present Day Dollars
Cumulative Costs (Millions)
0
1
2
3
4
5
6
7
8
9
10
11
2000
2010
2020
2030
2040
2050
2060
2070
Date (years)
Minimum Coating Treatment Initially, Recoat at Year 21
Complete Recoat Now Including Rails

Alternative Coating Scenarios for Old Spences Bridge Costs are Present Day Dollars

Time (yr)	Year	Minimum Coating Treatment Initially, Further work at Year 11,	Cost	Cumulative Cost	Complete Recoat Now Including Rails	Cost	Cumulative Cost		Cost	Cumulative Cost		Cost	Cumulative Cost
0	2009			0			0			0			0
1	2010	Touchup 25% of structure including railings to stop corrosion	406,157	406,157	Recoat All	5,966,069	5,966,069			0			0
2	2011			406,157			5,966,069			0			0
3	2012			406,157			5,966,069			0			0
4	2013			406,157			5,966,069			0			0
5	2014			406,157			5,966,069			0			0
6	2015			406,157			5,966,069			0			0
7	2016			406,157			5,966,069			0			0
8	2017			406,157			5,966,069			0			0
9	2018			406,157			5,966,069			0			0
10	2019			406,157			5,966,069			0			0
11	2020	Touchup 50%	812,315	1,218,472	Touchup 15%	243,694	6,209,763			0			0
12	2021			1,218,472			6,209,763			0			0
13	2022			1,218,472			6,209,763			0			0
14	2023			1,218,472			6,209,763			0			0
15	2024			1,218,472			6,209,763			0			0
16	2025			1,218,472			6,209,763			0			0
17	2026			1,218,472			6,209,763			0			0
18	2027			1,218,472			6,209,763			0			0
19	2028			1,218,472			6,209,763			0			0
20	2029			1,218,472			6,209,763			0			0
21	2030	Recoat All	5,966,069	7,184,540			6,209,763			0			0
22	2031			7,184,540			6,209,763			0			0
23	2032			7,184,540			6,209,763			0			0
24	2033			7,184,540			6,209,763			0			0
25	2034			7,184,540			6,209,763			0			0
26	2035			7,184,540			6,209,763			0			0
27	2036			7,184,540			6,209,763			0			0
28	2037			7,184,540			6,209,763			0			0
29	2038			7,184,540			6,209,763			0			0
30	2039			7,184,540	Overcoat All	3,610,305	9,820,068			0			0
31	2040	Touchup 20%	324,926	7,509,466			9,820,068			0			0
32	2041			7,509,466			9,820,068			0			0
33	2042			7,509,466			9,820,068			0			0
34	2043			7,509,466			9,820,068			0			0
35	2044			7,509,466			9,820,068			0			0
36	2045			7,509,466			9,820,068			0			0
37	2046			7,509,466			9,820,068			0			0
38	2047			7,509,466			9,820,068			0			0
39	2048			7,509,466			9,820,068			0			0
40	2049			7,509,466			9,820,068			0			0
41	2050			7,509,466			9,820,068			0			0
42	2051			7,509,466			9,820,068			0			0
43	2052			7,509,466			9,820,068			0			0
44	2053			7,509,466			9,820,068			0			0
45	2054			7,509,466			9,820,068			0			0
46	2055			7,509,466			9,820,068			0			0
47	2056			7,509,466			9,820,068			0			0
48	2057			7,509,466			9,820,068			0			0
49	2058			7,509,466			9,820,068			0			0
50	2059			7,509,466			9,820,068			0			0
Resulting Coating Condition		This treatement immediately touches up the worst 25% of the failed coating. At year 10 (2020) additional touchup is done to rehabilitate the areas originally coated plus new areas that have deteriorated. At year 21 (2030) a full recoat is done to completely rehabilitate the coating. With a touchup at year 31 (2040) this coating will last until year 50 (2059). This option stops any further loss of steel due to corrosion. At the end of 50 years, the coating will need an overcoat in the next 5 years or so.			This scenario recoats the entire structure immediately, then applies minimum coating treatments to extend the life of the coating out to 50 years. There is no steel loss and the structure is aesthetically much more pleasing for the first 20 years . At the end of 50 years, the coating will be ready for a complete recoat in the next 10 years.								


Alternative Coating Scenarios for
Old Spences Bridge
Net Present Value Dollars at 4%
Cumulative Costs (Millions)
0
1
2
3
4
5
6
7
8
2000
2010
2020
2030
2040
2050
2060
2070
Date (years)
Minimum Coating Treatment Initially, Recoat at Year 21
Complete Recoat Now Including Rails

Alternative Coating Scenarios for Old Spences Bridge Net Present Value Dollars at 4%

Time (yr)	Year	Minimum Coating Treatment Initially, Further work at Year 11,	Cost	Cumulative Cost	NPV Cost	Complete Recoat Now Including Rails	Cost	Cumulative Cost	NPV Cost		Cost	Cumulative Cost	NPV Cost		Cost	Cumulative Cost	NPV Cost
0	2009											0	0				
1	2010	Touchup 25% of structure including railings to stop corrosion	406,157	406,157	390,536	Recoat All	5,966,069	5,966,069	5,736,604			0	0			0	0
2	2011			406,157	390,536			5,966,069	5,736,604			0	0			0	0
3	2012			406,157	390,536			5,966,069	5,736,604			0	0			0	0
4	2013			406,157	390,536			5,966,069	5,736,604			0	0			0	0
5	2014			406,157	390,536			5,966,069	5,736,604			0	0			0	0
6	2015			406,157	390,536			5,966,069	5,736,604			0	0			0	0
7	2016			406,157	390,536			5,966,069	5,736,604			0	0			0	0
8	2017			406,157	390,536			5,966,069	5,736,604			0	0			0	0
9	2018			406,157	390,536			5,966,069	5,736,604			0	0			0	0
10	2019			406,157	390,536			5,966,069	5,736,604			0	0			0	0
11	2020	Touchup 50%	812,315	1,218,472	918,200	Touchup 15%	243,694	6,209,763	5,894,904			0	0			0	0
12	2021			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
13	2022			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
14	2023			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
15	2024			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
16	2025			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
17	2026			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
18	2027			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
19	2028			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
20	2029			1,218,472	918,200			6,209,763	5,894,904			0	0			0	0
21	2030	Recoat All	5,966,069	7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
22	2031			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
23	2032			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
24	2033			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
25	2034			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
26	2035			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
27	2036			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
28	2037			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
29	2038			7,184,540	3,536,311			6,209,763	5,894,904			0	0			0	0
30	2039			7,184,540	3,536,311	Overcoat All	3,610,305	9,820,068	7,008,028			0	0			0	0
31	2040	Touchup 20%	324,926	7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
32	2041			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
33	2042			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
34	2043			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
35	2044			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
36	2045			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
37	2046			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
38	2047			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
39	2048			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
40	2049			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
41	2050			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
42	2051			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
43	2052			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
44	2053			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
45	2054			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
46	2055			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
47	2056			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
48	2057			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
49	2058			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
50	2059			7,509,466	3,632,639			9,820,068	7,008,028			0	0			0	0
Resulting Coating Condition		This treatement immediately touches up the worst 25% of the failed coating. At year 10 (2020) additional touchup is done to rehabilitate the areas originally coated plus new areas that have deteriorated. At year 21 (2030) a full recoat is done to completely rehabilitate the coating. With a touchup at year 31 (2040) this coating will last until year 50 (2059). This option stops any further loss of steel due to corrosion. At the end of 50 years, the coating will need an overcoat in the next 5 years or so.				This scenario recoats the entire structure immediately, then applies minimum coating treatments to extend the life of the coating out to 50 years. There is no steel loss and the structure is aesthetically much more pleasing for the first 20 years . At the end of 50 years, the coating will be ready for a complete recoat in the next 10 years.											

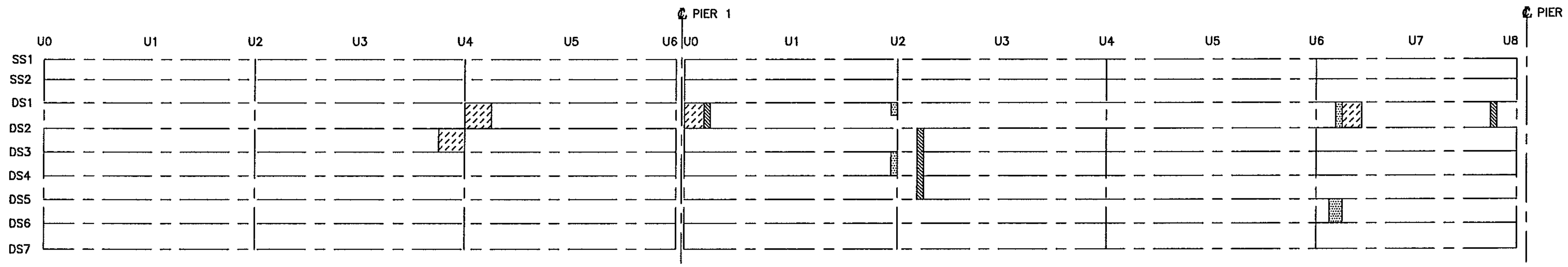
Appendix E
Condition Survey of Deck Soffit

LEGEND

DELAM

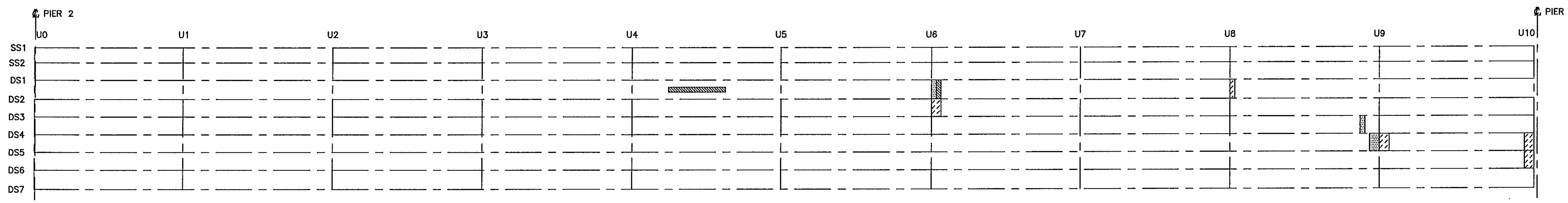
EXPOSED REBAR

CRACK



SPAN 1 – PLAN
SCALE 1:100

SPAN 2 – PLAN
SCALE 1:100



SPAN 3 – PLAN
SCALE 1:100

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Rev	Date	Description	Init
PA	09/11/20	DRAFT	

R E V I S I O N S

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE
DELAMINATION LOCATION – 1

PREPARED UNDER THE DIRECTION OF

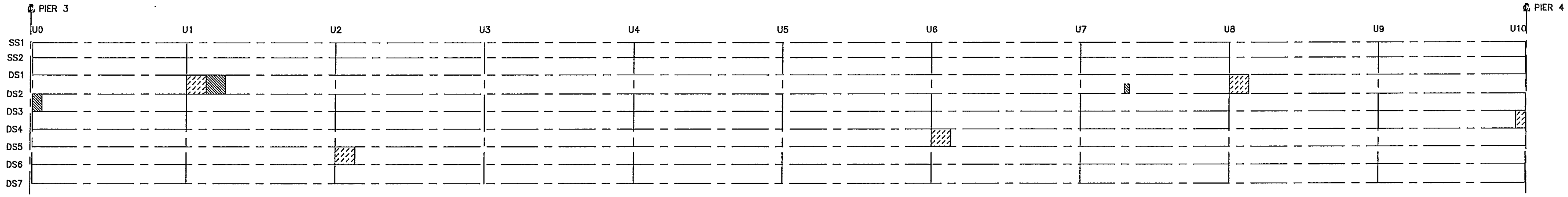
ENGINEER OF RECORD

DATE

SEAL

DESIGNED TD DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

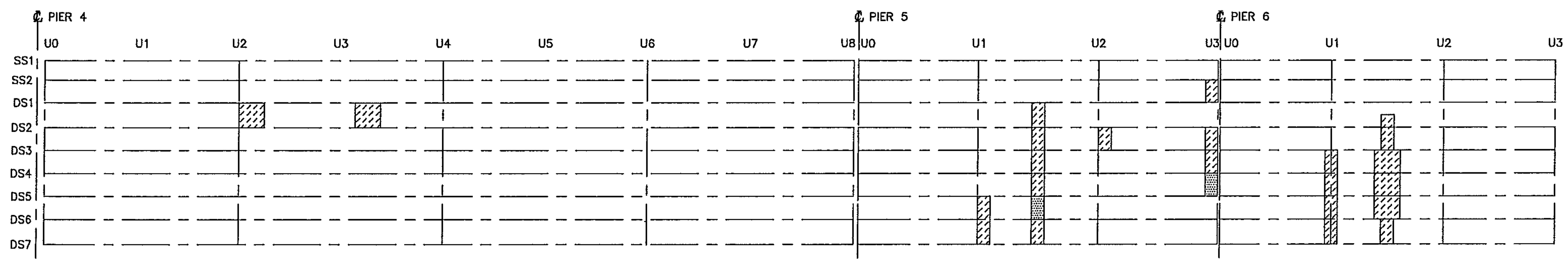
FILE No. 1884
PROJECT No. 2411
REG. 2
DRAWING No. 1884-2411-R05 PA



SPAN 4 – PLAN
SCALE 1:100

LEGEND

- DELAM
- EXPOSED REBAR
- CRACK



SPAN 5 – PLAN
SCALE 1:100

SPAN 6 – PLAN
SCALE 1:100

SPAN 7 – PLAN
SCALE 1:100

Consultant Logo

BUCKLAND & TAYLOR LTD.
Bridge Engineering

Rev	Date	Description	Init
PA	09/11/20	DRAFT	

REVISIONS

BRITISH COLUMBIA
Ministry of Transportation & Infrastructure
Southern Interior Region

THOMPSON NICOLA DISTRICT
OLD SPENCES BRIDGE No. 2411

OLD SPENCES BRIDGE DELAMINATION LOCATION – 2

PREPARED UNDER THE DIRECTION OF

ENGINEER OF RECORD

DATE

SEAL

DESIGNED TD DATE 2009-11-20
CHECKED AG DATE 2009-11-20
DRAWN KAM DATE 2009-11-20
SCALE AS NOTED
NEGATIVE No.

FILE No.	PROJECT No.	REG.	DRAWING No.
1884	2411	2	1884-2411-R06 PA

File: G:\1884\Old Spences Bridge\Inspection Report\Drawings\1884-2411-R06.dwg Save Date: Nov 17, 2009 9:14 AM Saved By: KMARAT

Appendix F
Record of Observations made on the East and West Railings

West Railing Observations:
Typical Condition: Minor coating failure with light surface corrosion. No section loss.

Span	Panel	Comment
Span 1	1	Typical Condition
	2	Typical Condition
	3	Typical Condition
	4	Typical Condition
	5	Typical Condition
	6	Missing connection pins in lattice frame; top north corner
Span 2	1	Typical Condition
	2	Typical Condition
	3	Typical Condition
	4	Typical Condition
	5	Typical Condition
	6	Typical Condition
	7	Typical Condition
	8	Typical Condition
Span 3	1	Corrosion damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
	2	Corrosion damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
	3	Typical Condition
	4	Typical Condition
	5	Typical Condition
	6	Typical Condition
	7	Typical Condition
	8	Corrosion damage to corner gusset at base of lattice panel.
	9	Typical Condition
	10	Corrosion damage to corner gusset at base of lattice panel.
	11	Typical Condition
	12	Corrosion damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
	13	Typical Condition
	14	Typical Condition
	15	Typical Condition
	16	Typical Condition
	17	Typical Condition
	18	Typical Condition
	19	Typical Condition
	20	Typical Condition
Span 4	1	Typical Condition with heavier lattice corrosion.
	2	Typical Condition with heavier lattice corrosion.
	3	Typical Condition with heavier lattice corrosion.

	4	Corrosion damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
	5	Typical Condition with heavier lattice corrosion.
	6	Corrosion and mechanical damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
	7	Typical Condition with heavier lattice corrosion.
	8	Typical Condition with heavier lattice corrosion.
	9	Typical Condition with heavier lattice corrosion.
	10	Typical Condition with heavier lattice corrosion.
	11	Typical Condition with heavier lattice corrosion.
	12	Typical Condition with heavier lattice corrosion.
	13	Typical Condition with heavier lattice corrosion.
	14	Typical Condition with heavier lattice corrosion.
	15	Typical Condition with heavier lattice corrosion.
	16	Typical Condition with heavier lattice corrosion.
	17	Typical Condition with heavier lattice corrosion.
	18	Typical Condition with heavier lattice corrosion.
	19	Typical Condition with heavier lattice corrosion.
	20	Corrosion and mechanical damage to corner gusset at base of lattice panel. Pack rust has popped the connection.
Span 5	1	Typical Condition
	2	Typical Condition
	3	Typical Condition
	4	Loose fence panel at base
	5	Typical Condition
	6	Typical Condition
	7	Typical Condition
	8	Typical Condition
Span 6	1	Minor warping of panel lattice.
	2	Typical Condition
	3	Typical Condition
Span 7	1	Typical Condition
	2	Typical Condition
	3	Typical Condition

East Railing Observations:
Typical Condition: Minor Coating Failure with light surface corrosion. No section loss

Span	Panel	Comment
Span 1	1	Collision damage, bent lacing and not connected to the railing post
	2	Not connected to the railing post. Panel is bent due to collision
	3	Minor Coating failure and light corrosion
	4	Bent out slightly. Minor damage to lacing
	5	Same as panel 3
	6	Areas of missing lacing. Collision damage
Span 2	1	Collision damage. Bent lacing
	2	Typical Condition
	3	Typical Condition
	4	Minor scrapes and dents in lacing. Broken Connection to post
	5	Loose connection to south post at bottom rail
	6	Typical Condition
	7	Typical Condition
	8	Rusted hole in corner gusset
Span 3	1	Typical Condition
	2	Typical Condition. Minor Scrapes but no gouges. Possible due to snow plows
	3	Typical Condition
	4	Typical Condition
	5	Pack rust in corner gusset between gusset and rail member
	6	Typical Condition. Minor scrapes
	7	Typical Condition
	8	Typical Condition
	9	Occasional missing fasteners in lacing
	10	Typical Condition. Signs of previous repairs
	11	Signs of previous repairs. Minor vehicle damage
	12	Bent lacing. Otherwise typical conditions
	13	Lacing not connected to bottom rail in two locations
	14	Bent and broken lacing. Loose connection at bottom rail on north end
	15	Typical Condition
	16	Typical Condition
	17	Typical Condition
	18	Typical Condition
	19	Typical Condition
	20	Pack rust at north end of bottom rail gusset plate. Has not yet popped connection

Span 4	1	Typical Condition
	2	Pack rust at north end of bottom rail gusset plate. Has not yet popped connection
	3	Typical Condition
	4	Pack rust at north end of bottom rail gusset plate. Has not yet popped connection
	5	Pack rust at south end of bottom rail gusset.
	6	Minor collision or plow damage
	7	Loose connection at south end of bottom rail
	8	Typical condition. Pack rust at bottom rail gusset on north end of rail
	9	Typical Condition
	10	Typical Condition
	11	Pack rust at south end of bottom rail gusset. Loose lacing members
	12	Minor vehicle impact damage
	13	Pack rust at south end of bottom rail gusset.
	14	Minor vehicle impact damage
	15	Typical Condition
	16	Minor vehicle impact damage. Pack rust has popped the gusset
	17	Top gusset has been repaired. Typical Condition
	18	One loose lacing bar at bottom rail. Pack rust at north end of bottom rail has popped rivets
	19	Pack rust at south end of rail connection. Typical Condition
	20	Typical Condition
Span 5	1	Typical Condition
	2	Missing bolt in bottom rail connection to post at south end. Typical condition
	3	Numerous bent and damaged lacing bars. The railing is loose and rattles. Connection at South end of the bottom rail is loose.
	4	Raised rivets on the top rail at south post. Otherwise typical condition
	5	Typical Condition
	6	Typical Condition
	7	Typical Condition
	8	Rust jacking and warping of gusset at base of fence panel.
Span 6	1	Rust jacking of gusset at base of fence panel. Rattles when struck.
	2	Typical Condition
	3	Typical Condition
Span 7	1	Typical Condition
	2	Rusted hole in corner gusset at base of panel
	3	Typical Condition

Post Observations – West Railing

Span	Post	Comment
Span 1	P1	Compromised anchor bolts and damaged base plate. Cracks in base plate at NW,SW and SE corner of post. Concrete under the post is spalled and undermining the base plate.
	P2	No significant issues observed. A U-bolt has been welded around the post at the base.
	P3	No significant issues
	P4	Areas of minor coating failure. No significant issues – Typical
	P5	Typical
	P6	Typical
	P7	Typical
Span 2	P1	Typical. 2 improperly seated rivets
	P2	Typical. Requires new paint
	P3	Typical 2 questionable rivet heads in vertical angle. Not a concern
	P4	Typical
	P5	Typical
	P6	Typical
	P7	Typical
	P8	Typical
Span 3	P1	Typical
	P2	Typical
	P3	Typical
	P4	Typical
	P5	Typical. Coating on the back gusset is bubbled and cracked
	P6	Typical
	P7	Typical. Coating on the lower gusset plate (north face) has failed
	P8	Typical. Coating on the lower gusset plate (north face) has failed
	P9	Typical. Back gusset has been marked with an X
	P10	Typical
	P11	Typical
	P12	Bubbled and cracked paint on the north face of the back gusset
	P13	Typical. Coating on the lower gusset plate (north face) has failed
	P14	Typical
	P15	Typical
	P16	Typical
	P17	Minor corrosion buildup on the lower gusset. Back gusset marked with an X
	P18	Typical
	P19	Typical
	P20	Typical
	P21	Typical
Span 4	P1	Lower gusset has corrosion buildup estimated at approx 5 mm thick
	P2	Typical
	P3	Lower gusset has corrosion buildup estimated at approx 5 mm thick
	P4	Typical

	P5	Typical
	P6	Typical
	P7	Typical
	P8	Typical
	P9	Typical
	P10	Typical
	P11	Back gusset plate marked with an X
	P12	Typical
	P13	Typical
	P14	Typical
	P15	Typical
	P16	Typical
	P17	Minor pitting on lower gusset on the north face
	P18	Paint peeling on the north face of the lower gusset
	P19	Typical
	P20	Typical
	P21	Typical
Span 5	P1	Typical
	P2	Typical
	P3	Typical. 2 rivets to the top railing are not set flush to the top of the rail.
	P4	Typical
	P5	Typical
	P6	Typical Condition
	P7	Typical Condition
	P8	Misalignment over expansion joint
Span 6	P1	Misalignment over expansion joint
	P2	Typical Condition
	P3	Typical Condition
Span 7	P1	Typical Condition
	P2	Typical Condition
	P3	End post leaning away from bridge causing deformation in connections. Pins popped out at the top connection.

Post Observations – East Railing

Span	Post	Comment
Span 1	P1	Overall good condition
	P2	Typical Condition
	P3	Advanced corrosion with minor pitting
	P4	Typical Condition
	P5	Typical Condition
	P6	Typical Condition
	P7	Typical Condition
Span 2	P1	Typical Condition
	P2	Advanced corrosion
	P3	Advanced corrosion
	P4	Typical Condition
	P5	Typical Condition
	P6	Typical Condition
	P7	Typical Condition
	P8	Typical Condition
Span 3	P1	Typical Condition
	P2	Typical Condition
	P3	Typical Condition
	P4	Typical Condition
	P5	Typical Condition
	P6	Typical Condition
	P7	Typical Condition
	P8	Typical Condition
	P9	Typical Condition
	P10	Typical Condition
	P11	Typical Condition
	P12	Missing bolt at base connection of fence panel.
	P13	Typical Condition
	P14	Corrosion sheeting of horizontal support member with minor pitting.
	P15	Loose bolt at base connection of fence panel.
	P16	Typical Condition
	P17	Typical Condition
	P18	Corrosion sheeting of horizontal support member with minor pitting.
	P19	Typical Condition
	P20	Typical Condition
	P21	Typical Condition
Span 4	P1	Typical Condition
	P2	Typical Condition
	P3	Typical Condition
	P4	Advanced corrosion

	P5	Typical Condition
	P6	Typical Condition
	P7	Typical Condition
	P8	Typical Condition
	P9	Typical Condition
	P10	Typical Condition
	P11	Typical Condition
	P12	Typical Condition
	P13	Typical Condition
	P14	Typical Condition
	P15	Typical Condition
	P16	Typical Condition
	P17	Typical Condition
	P18	Typical Condition
	P19	Typical Condition
	P20	Typical Condition
	P21	Typical Condition
Span 5	P1	Typical Condition
	P2	Main vertical post consists of bent angles and buckled plates.
	P3	Typical Condition
	P4	Loose bolt at base connection of fence panel.
	P5	Loose bolt at base connection of fence panel.
	P6	Dime sized hole in horizontal angle support
	P7	Typical Condition
	P8	Missing bolt at base of connection to fence panel.
Span 6	P1	Typical Condition
	P2	Typical Condition
	P3	Missing bolt at base of connection to fence panel.
Span 7	P1	Missing bolt at base of connection to fence panel.
	P2	Typical Condition
	P3	Typical Condition