The Old Skeena River Bridge
Rehabilitation and Recoating of a 100-Year-Old Steel Truss

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Abstract

The Old Skeena River Bridge is a 9-span, 341 m long, single lane bridge consisting of 3 steel stringer approach spans, 5 steel deck truss spans and a steel through truss span, crossing over the Skeena River in Terrace, BC. The structure opened on July 21, 1925, was realigned to accommodate a parallel railway bridge in 1953, and the timber deck was replaced with an open grate steel deck in 2001. The bridge was last recoated in the early 1980s and, at the time of the rehabilitation, still contained red-lead primer.

The project included:

- Close proximity inspection with ropes and ladders (2017) and snooper truck (2021).

- Load evaluation of trusses for verification of existing posted load restrictions and ability to carry the BC Ministry of Transportation and Infrastructure (Ministry) snooper truck inspection vehicle,
- Evaluation of construction lateral wind load capacity during enclosed coating operations.
- Steel coating renewal considering various coating systems and methodologies (overcoat vs. removal and recoating) combined with Life-Cycle Cost Analysis of the different options.
- Installation of 6 new sidewalk refuge bays to accommodate pedestrian movements along the bridge.
- Replacement of corroded steel roller bearings with sliding elastomeric bearings, including strengthening to facilitate bridge jacking operations for bearing replacement.
- Replacement or repair of severely corroded members and plates.

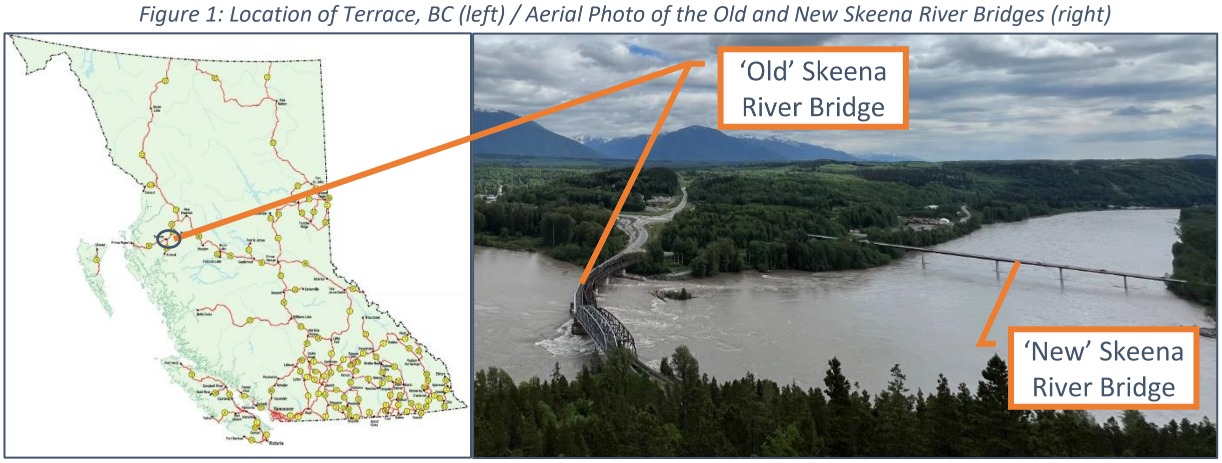
There were numerous challenges and lessons learned during the design and construction process, including:
- Inspection access for truss bridges and determining appropriate levels of expenditure on inspection during the design process.
- Responding to excessive corrosion uncovered following sandblasting.
- Considerations for repairs of historic members – rivet removal and bolt installation.
- Repair details developed during construction for ‘primary’ (truss diagonals and tension chords) and ‘secondary’ (floor beams, stringers, gusset plates and bracing) members.
- Appropriate contingency allowances during construction and cooperation with the Contractor.

This paper describes the project along with solutions and responses to challenges encountered during construction, culminating in several lessons learned that can be applied to future projects.

Background

The Old Skeena River Bridge (No. 00473) is located on Old Highway 16 (Lakelse Avenue) spanning over the Skeena River in Terrace, BC. It is located just 650 m northwest of the junction of Highway 16 and Highway 37, and it accommodates single lane-alternating traffic with traffic lights on timers at both ends of the structure.

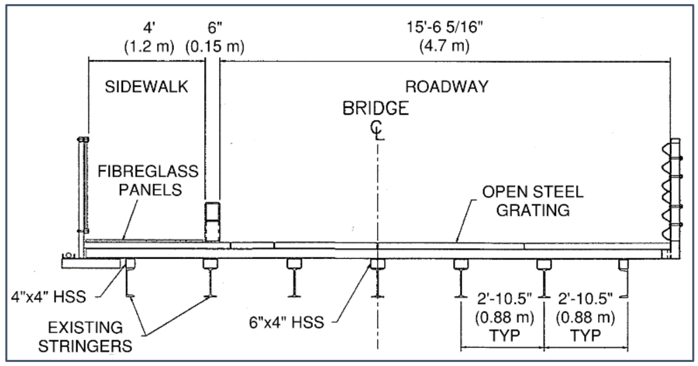
Figure 1: Location of Terrace, BC (left) / Aerial Photo of the Old and New Skeena River Bridges (right)



Source: BC Ministry of Transportation and Infrastructure (left)

Total bridge length is approximately 1126 feet (343 m) and is generally on a horizontally curved alignment with straight segments kinked at the piers. The deck is generally 15'-5.5" (4.7 m) wide with a 4' (1.2 m) sidewalk and narrows to 13'-4" (4.1 m) wide on the through truss with a 3'-7" (1.1 m) sidewalk (see Figure 2).

Figure 2: Typical Roadway and Sidewalk Section Following the 2001 Rehabilitation

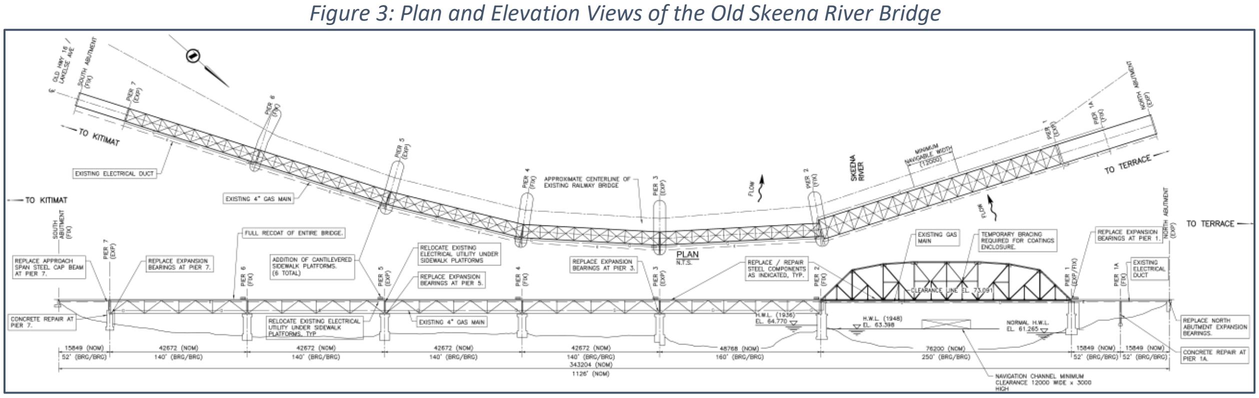


Source: BC Ministry of Transportation & Infrastructure

The bridge's current configuration (see Figure 3) has nine (9) spans, consisting of:

- One 50' (15.2 m) steel stringer south approach span
- Four 140' (42.7 m) steel deck truss spans
- One 160' (48.8 m) steel deck truss span
- One 250' (76.2 m) steel through truss main span
- Two 50' (15.2-15.2 m) steel stringer north approach spans.

Figure 3: Plan and Elevation Views of the Old Skeena River Bridge



The piers and abutments are made of concrete and are founded on bedrock.

The bridge was originally constructed in 1924 and opened to traffic in 1925 with a timber deck and timber approach trestles. Although the six (6) steel truss spans are original, the structure has undergone major reconstruction / rehabilitations over its lifetime, including:

- In 1953, the bridge was shifted slightly from its original alignment to accommodate the curved alignment of a parallel CN Railway structure located immediately west of the bridge. The concrete piers were widened (see Figure 4) to accommodate both the original bridge and the railway bridge on the new alignment. The new alignment resulted in four (4) kinks along the bridge (Piers 2, 3, 4 and 5) compared to the original single large kink at the transition of the through truss to deck truss spans (Pier 2 only).
- In 1963, a design was completed to twin the existing bridge and install a new two-lane concrete deck. This design would have required further widening of the piers and replacement of the through truss with a steel girder span. This design was never constructed.
- In the early 1980s, the bridge was recoated with the colour changing from black to silver.
- In 2001, an open grate steel deck system was installed which replaced the previous timber decking (see Figure 5).

Figure 4: Widened Concrete Piers prior to Realignment to Accommodate CN Railway Bridge

Source: Kitimat Museum and Archives

Figure 5: 2001 Steel Deck System from Above (left) and Below (right). Decking Supported on Existing Stringers and Floor Beams

The design vehicle noted on the bridge realignment drawings from 1953, showed a concentrated 20 ton (18.2 tonne / 178 kN) design vehicle and a uniform load of 92 lbs/ft^2 (4.4 kPa). An impact allowance of 30% was applied on the wheel loads. Following a load evaluation in 2002, the existing bridge was load restricted to a 4,000 kg Gross Vehicle Weight (GVW), the vehicle length was restricted to 7 m, and the travel speed was posted for a maximum of 30 km/hr (see Figure 6).

Figure 6: Current Bridge Load Restrictions

Routine inspections of the bridge indicated varying light to heavy corrosion on all members in addition to failed roller bearings at expansion piers.

Project Objectives

The project followed the Canadian Highway Bridge Design Code (CHBDC) CSA S6:19[1] and the BC Ministry of Transportation and Infrastructure Supplement to S6:19[2] and aimed to achieve the following general objectives:

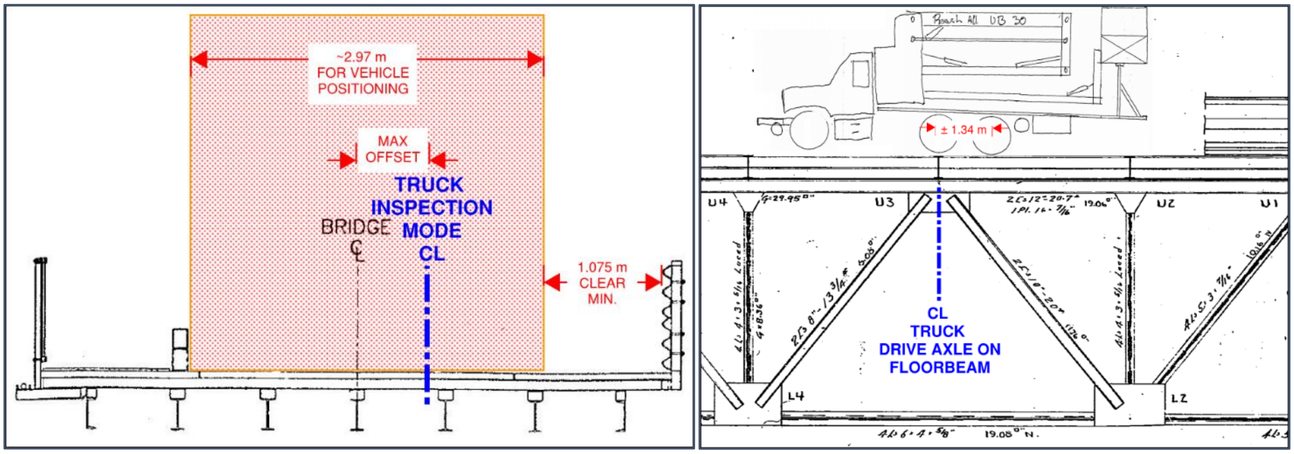
- The bridge will continue to operate as a single-lane-alternating structure with traffic light controls at each end.
- The bridge will continue to be posted at a 4,000 kg (4.4 ton) GVW as it has been since 2004.
- It would be beneficial to allow for the use of the Ministry's 30 ton (inspection vehicle on future inspection work.
- The primary objective is to extend the service life of the bridge as much as possible. The bridge required a structural intervention before it became damaged beyond repair.

Project Design Components

Updated Load Evaluation

It was determined that one of the critical elements of the reduced load capacity of the bridge was the vehicle loads on the external channel deck stringers. A strategy was prepared to allow a snooper truck on to the bridge by restricting the wheel path to the middle of the bridge to avoid loading up the exterior stringers. Additionally, during snooper operations, the main axles of the snooper truck were parked over top of floor beams to reduce the loading on the stringers. Finally, a lighter snooper truck was located which was only 23.2 tons (21,100 kg), or roughly 70% of the Ministry snooper truck (see Figure 7).

Figure 7: Snooper Truck Inspection Strategy for Horizontal (left) and Longitudinal (right) Positioning



Close Proximity Inspection

Following the load restrictions in 2004, structural condition inspections between 2005 and 2016 were performed with ropes and ladders but could not reach all components. Following the re-evaluation of the loading capacity, a snooper truck inspection was able to be performed in 2021. It was anticipated that 3 days would be sufficient to quantify the extent of structural repairs required, including a coatings inspection. The primary findings of the 2021 inspections included:

- Upper Chords were in worse condition than Lower Chords and the West side of the truss was in worse condition than the East side (the West side is adjacent to the CN Rail bridge).
- The existing coatings system had failed, and severe rust was found at multiple locations but section loss was mainly on secondary members. A small hole in a primary diagonal truss member was noted during the inspection (Figure 9, right).
- The quantity of components requiring repair approximately doubled following snooper truck inspection compared to the previous quantity identified from the 2016 rope and ladder access inspection (see Figure 8 and Figure 9).

Figure 8: Snooper Truck Inspection (left) and Top Chord Inspection (right)

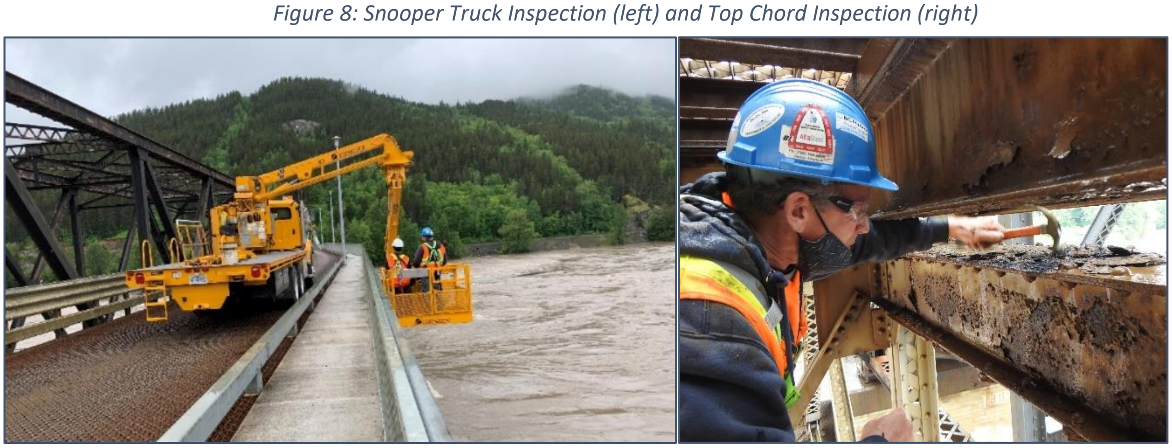


Figure 9: Examples of Severe Corrosion in Deck Stringer (left) and Truss Diagonal (right)

Replace Expansion Bearings

The original truss structure utilized 12 steel roller bearings at the expansion locations. All the rollers had corroded such that the bearings were unable to expand, although the existing fixed bearings were in fair condition. Details were developed to replace the roller bearings with sliding elastomeric bearings with lateral restrainers (see Figure 10).

Figure 10: Existing (Seized) Roller Bearings (left) and New Sliding Elastomeric Bearings with Lateral Restraint (right)

Repair / Replace Corroded Members

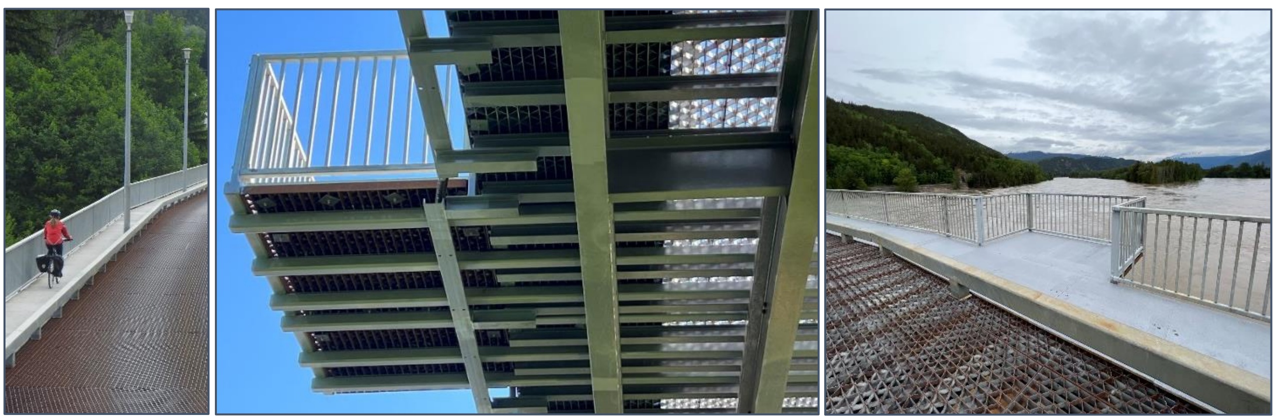
Details were developed to repair members identified during the initial condition inspection. The design repairs utilized galvanized components which could be prepared ahead of time (see Figure 11). During construction, several additional locations requiring repairs were identified once scaffolding access was provided and sandblasting had cleared off some of the excess surface corrosion. Initially, additional repair locations were repaired using galvanized sheets and angles cut on site, but eventually, plain steel sheets (prime coat applied on site) were used to avoid impacting the schedule and because everything would be covered during the full recoat of the bridge.

Figure 11: Typical Repairs at Lower Gusset (left), Floor Beam (centre) and Upper Gusset (right)

Sidewalk Refuge Bays

As a late addition to the design, 5' x 10' (1.5 m x 3.0 m) refuge bays were included at six locations along the sidewalk to facilitate passing along the existing narrow 4' (1.2 m) wide sidewalk. These bays were supported by cantilevered beams which were inserted between the first two deck stringers and the existing decking (see Figure 12).

Figure 12: Sidewalk Refuge Bays with Existing Narrow Sidewalk (left), Support Beams (centre) and Finished Bay (right)



Replace Cap Beam at Pier 7

The cap beam for the 50' (15.2 m) South approach span at Pier 7 was visibly warped and was one of the limiting members in the load capacity of the bridge. If fact, the 2021 snooper inspection had to access the bridge from the north side only and reverse off the bridge to avoid driving over Pier 7. A new cap beam was designed, including a temporary support beam with brackets for jacking, and the approach span stringers were seated on new elastomeric bearings (see Figure 13).

Figure 13: New South Approach Cap Beam and Elastomeric Bearings at Pier 7

Recoat Entire Structure

The existing coating system on the bridge, estimated to have been applied in the early 1980's, had failed and most members were experiencing mild to severe corrosion. The coating inspection confirmed the presence of Red Lead primer on the bridge. It was determined at the onset of the project that the only way to preserve the bridge was to perform a full recoat of the entire structure. There were several aspects of the recoating work.

Selection of Coating Type

Working with a coatings specialist, an options study was performed to select the most suitable and cost-effective coating type for the bridge. Life-cycle costing was performed for the following coating options:

- Option 1. Full Bridge Recoating using Organic Zinc / Epoxy / Polyurethane System
- Option 2. Recoating using Organic Zinc / Epoxy / Polyurethane System, staged over four (4) years
- Option 3. Overcoating using Calcium Sulphonate System
- Option 4. Recoating using Moisture Cured Urethane System
- Option 5. Overcoating using Moisture Cured Urethane System

Figure 14: Intermediate (Mid) Coat Product

Both overcoating options (Options 3 and 5) had the lowest net present value, however neither removed the existing lead paint. The estimated price difference between the Organic Zinc / Epoxy / Polyurethane system (Option 1) and the Moisture Cured Urethane system (Option 4) was approximately 3%, so the more conventional Organic Zinc / Epoxy / Polyurethane system was considered lower risk even though it has a slightly higher net present value. Staging the coating work over four years (Option 2) was considered too much inconvenience to the local traffic.

The actual coating product used on the bridge was a Sherwin Williams product, consisting of:

- Primer: Zinc Clad III
- Penetrating Sealer: Macropoxy 920 Pre-Prime
- Intermediate (Mid) Coat: Macropoxy 646 FC
- Top Coat: Acrolon 218 HS
- Caulking: Sikaflex 1A

Wind Loading

Due to the presence of lead paint and the ambient requirements for coating work, the bridge was required to be fully enclosed for containment of debris, temperature controls and humidity controls. The original trusses were not designed to carry the wind loading resulting from being fully enclosed. A 3D model of the structure was prepared and analysed for wind loads to determine the extent of enclosure permitted on each truss span.

The analysis determined that the structure was only able to accommodate partial enclosures and put limits on the maximum wind velocities that could be permitted during the enclosure. The 140’ and 160’ deck truss spans could only be half-enclosed, and the 250’ through truss span could only be roughly one-third enclosed (see Figure 15) and required temporary bracing to help distribute the lateral loads (see Figure 16). The analysis assumed that the member repairs to the truss horizontal bracing were performed prior to the full enclosure taking place. Work could be (and was) performed concurrently on adjacent spans which allowed the contractor to accelerate their schedule (see Figure 17).

Figure 15: Coating Staging for the Various Truss Spans and Allowable Wind Speeds During Construction

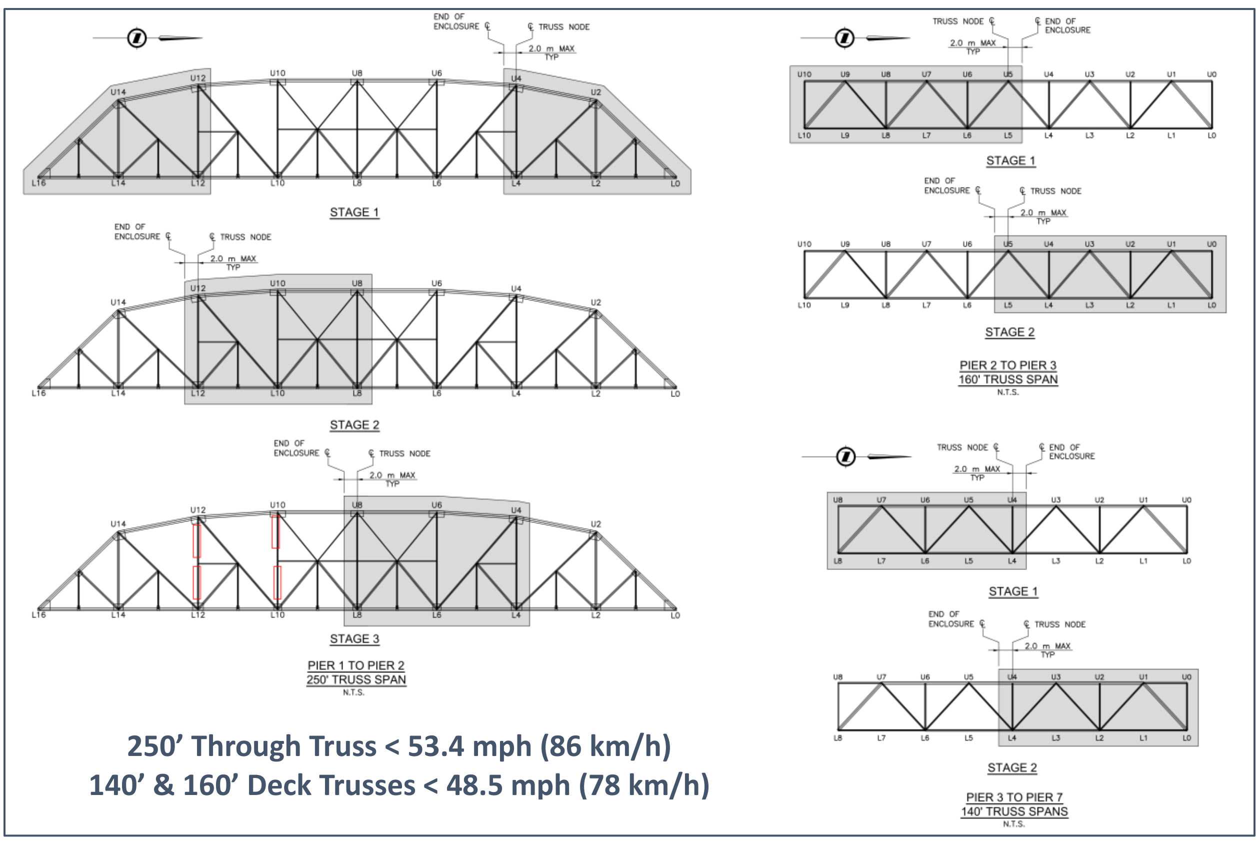


Figure 16: 250' Through Truss without Temporary Bracing (left) and with Temporary Bracing (right).
(Truss Deflections are Exaggerated to Illustrate Deflected Shape)

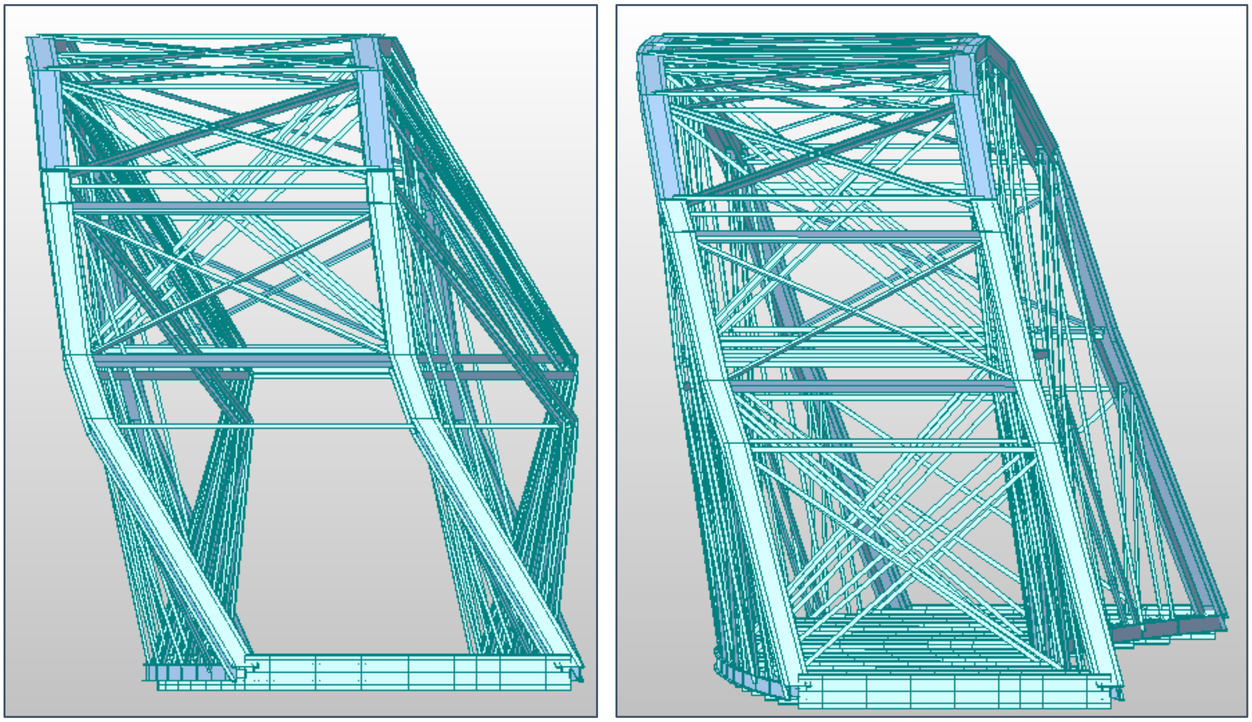


Figure 17: Coating Enclosure Work Occurring on Multiple Spans

Source: Gitga'at Park Derochie Industrial Services

Coating Methodology

Generally, the coating operations were performed after the structural repairs were complete and in the following order:

1) The enclosure was installed in accordance with SSPC Guide 6[3], Class 1A and 1W (see Figure 17). Engineered drawings of the scaffolding and enclosure were prepared by the Contractor.

2) The steel was blast cleaned to the stricter of SSPC-SP10[4] or the manufacturer's requirements. Enclosures were utilized to contain the steel shot system and harmful wastes during the blasting process (see Figure 18).
3) The coating is applied consisting of penetrating sealer, primer, stripecoat, intermediate (mid) coat and topcoat with quality assurance inspections occurring at each stage (see Figure 14).
4) Caulking was applied to seal the larger gaps following application of the topcoat in order to avoid schedule delays while waiting for the caulking to cure and also since the caulking was a close match to the final colour of the structure.
5) The enclosure was cleaned of any remaining debris and removed.

Figure 18: Steel Shot Blast Recycling Equipment (left) and Inside a Coatings Enclosure Following Top Coat Application (right)

Project Challenges

<u>Construction Loading Restrictions</u>

Due to the complexity of the multi-span truss structure and the extensive deterioration of many of the members, the structural analysis was time-consuming and required extensive familiarity with the structure condition. Considering the time and expense of having every bidding contractor required to assess the structure for their own construction methodologies, the design team attempted to provide clear loading requirements to the contractors such that they could avoid a detailed analysis if they stayed within the stated parameters. Specifically:

- Access scaffolding and coating enclosure loading assumptions were stated on the drawings. In the event that the contractor required the load limits to be exceeded, they would be required to submit an engineered load evaluation which demonstrated that the structure had adequate capacity (see Figure 19).
- It was stated that scaffolding and containment loading may not impose any bending moment into the truss members.
- Maximum wind pressures were stated on the drawings and were also converted into equivalent wind speeds for ease of comparison. The contractor was required to operate real-time wind monitoring equipment and to have a contingency plan to rapidly remove any containment if the required wind speeds were exceeded.

Figure 19: Allowable Construction Loading Stated on the Design Drawings

6.0 — CONSTRUCTION NOTES

1. THE FOLLOWING CONSTRUCTION LOAD LIMITS SHALL NOT BE EXCEEDED:

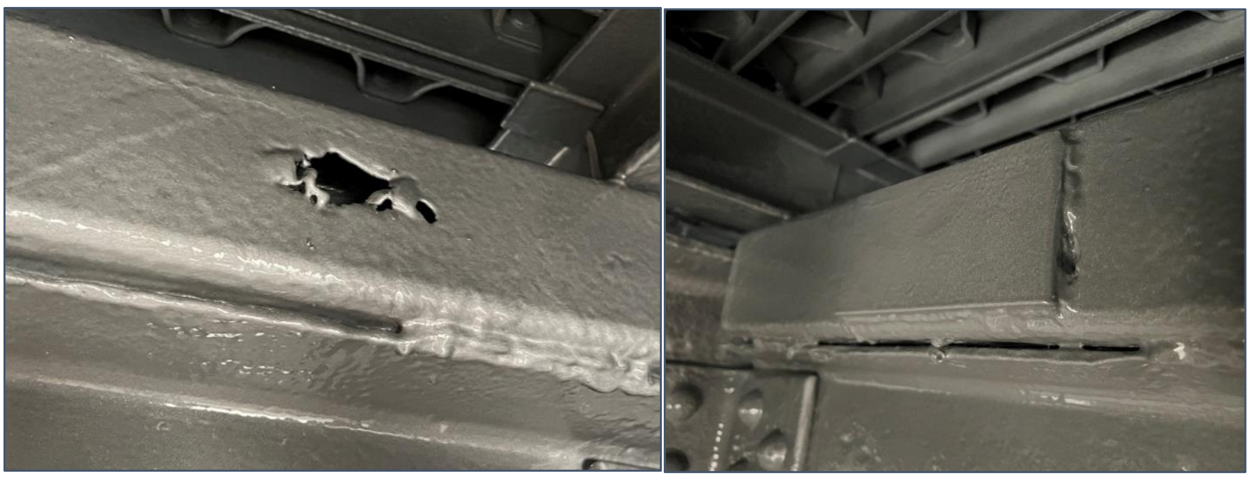
CONSTRUCTION DEAD LOAD:	1.35 kPa FOR 140 ft AND 160 ft SPANS AND APPROACHES**
	2.70 kPa FOR 250 ft THROUGH TRUSS SPAN**
CONSTRUCTION LIVE LOAD:	1.20 kPa**
DEBRIS LOAD ON SCAFFOLDING:	1.00 kPa**

Blast Cleaning Reveals

Blast cleaning revealed some significant additional member deterioration that wasn't observed during the previous inspections. This included:

- HSS members which were part of the steel decking system from 2001 which had been corroding from the inside, resulting in several perforations following blast cleaning (see Figure 20). These were repaired with a cover plate, and fortunately, they were mainly only present on the south approach span.

Figure 20: HSS Perforations Exposed by Blast Cleaning (left) and Cover Plate Repair (right)



- Truss diagonal chords and tension (bottom) chords revealed large openings at several locations (see Figure 21 and Figure 22) which were not fully revealed until after blast cleaning and coating operations. Compare Figure 21 with the much smaller 'pre-blast clean' holes shown in Figure 9. Once revealed, a rapid analysis was performed to determine if construction could continue, followed by further analysis to determine a repair methodology which considered maintaining integrity of the structural load path.
- Additional damage and perforations to several floor beams and stringers was revealed following blast cleaning (see Figure 20). The damage on some stringers was so significant that it was more cost-effective to replace the entire stringer than to implement the stringer repair design.
- The bearing stiffeners at the ends of several floor beams had deteriorated such that there was no contact between the stiffeners and the bottom flange (see Figure 23). Due to space constraints for making the repairs, a load capacity analysis was performed which demonstrated that a single replacement stiffener could provide adequate bearing capacity for the floor beams compared to replacing all four existing bearing stiffeners.

Figure 21: Large Holes in Primary Diagonal Members (left) and Completed Repairs (right)

Figure 22: Large Holes in Primary Tension Member (left) and Completed Typical Repair (right)

Figure 23: Post-Blast Clean Damage to Stringer (left) and Pre-Blast Clean Floor Beam Bearing Stiffeners (right)

Additional Member Repairs

Including to the Blast-Cleaning Reveals, the total quantity of steel repairs increased by over two times the originally estimated quantity. At the onset of construction, the contractor proposed to stockpile additional repair material in order to minimize potential supply delays during construction while waiting to perform the repairs. The design team worked with the contractor to develop repair details which utilized readily available plate thicknesses and common angles such that the plates could be cut and drilled on site to fit the specific repair location (see Figure 24).

Figure 24: Steel Sheeting Cut On Site for Additional Member Repairs

Initially, the stockpiled material was galvanized in accordance with the project specifications, but after the initial stockpile had been exhausted, plain steel (primed) was permitted to be used because the repairs were going to be recoated with the entire bridge anyway, and it was preferable that the coating work was not delayed.

Jacking for Bearing Replacement

The existing trusses were not designed for jacking for bearing replacement. The bearing replacement design included jacking modifications consisting of new gusset plates and / or additional stiffeners. Design assumptions considered that the secondary bracing members of the truss would provide some lateral support when the bridge was jacked. During jacking operations for the deck truss spans, localized corrosion of the secondary bracing members resulted in visible deformations during jacking (see Figure 25). Jacking was halted, and supplemental bracing was designed and installed to prevent further deformations from occurring.

Figure 25: Deterioration of Secondary Bracing Resulting in Observed Deformations During Jacking

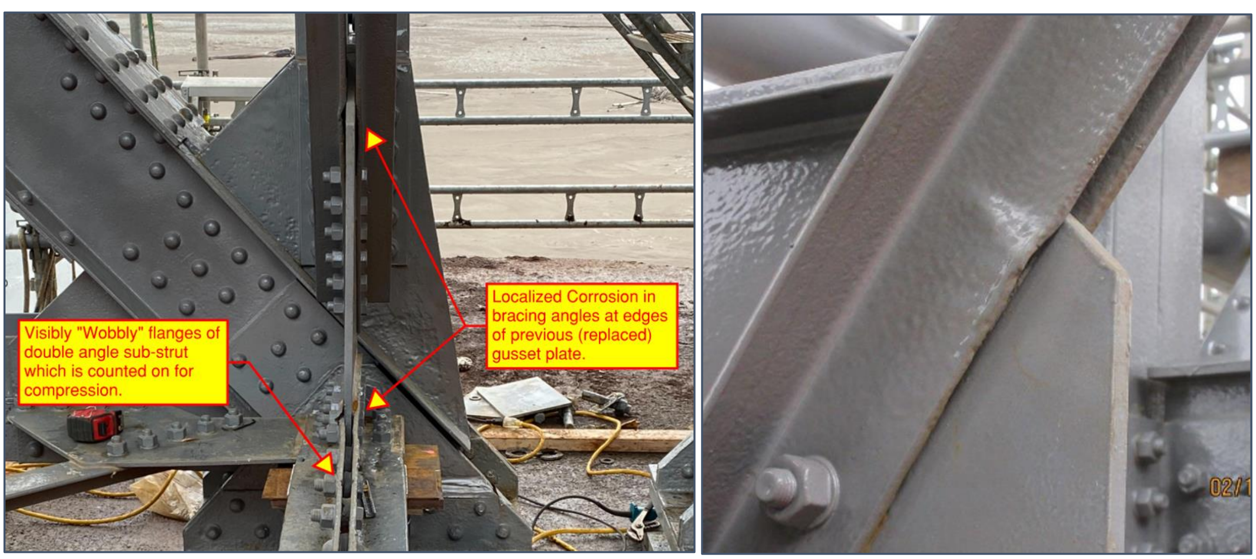
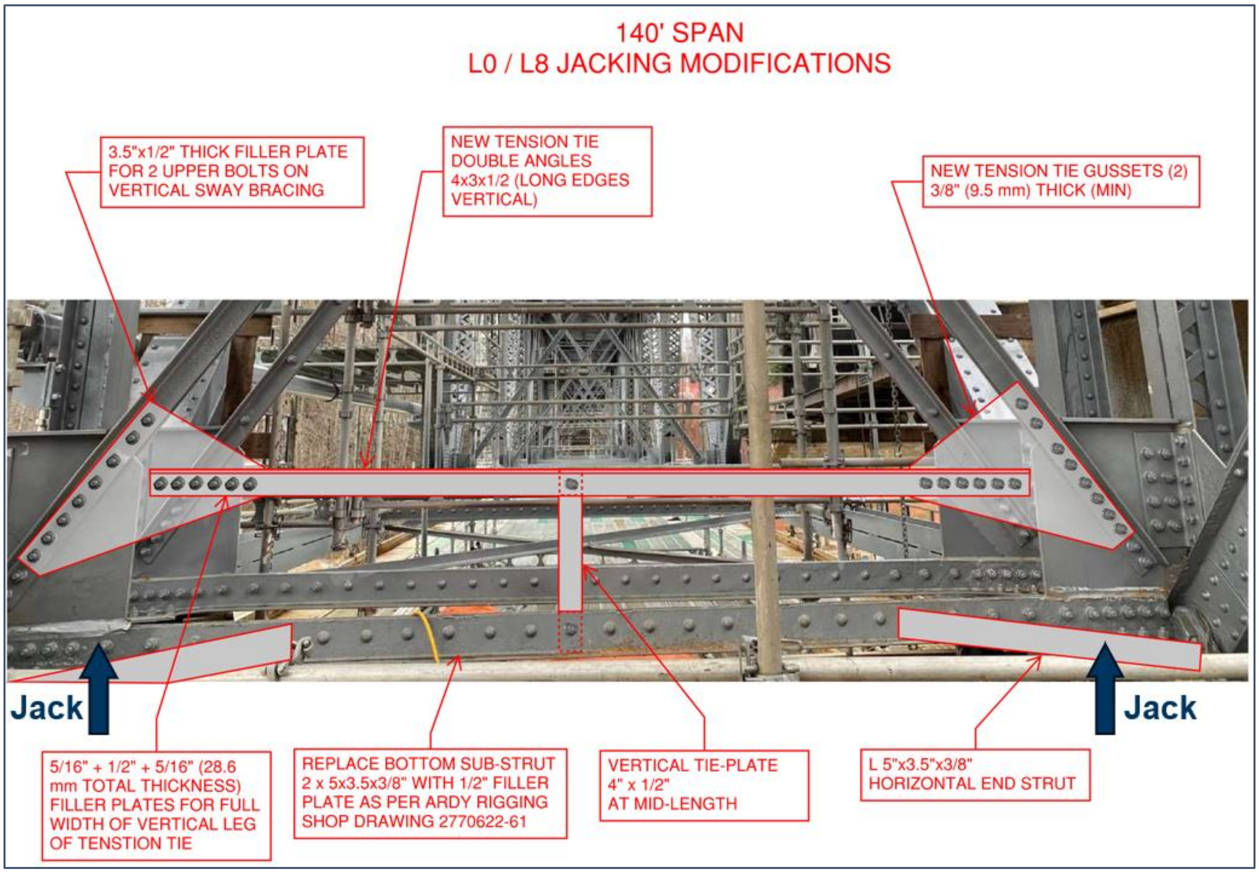


Figure 26: Additional Bracing Installed During Construction to Prevent Jacking Deformations



Lessons Learned

There were numerous challenges & lessons learned during the design & construction process, including:

1) It may be a net cost savings to invest in inspection during the design process. The more certainty you can have, the better. If a single member repair can cost in the order of $30k, the justification to spend some additional time performing a comprehensive inspection is likely worth it. Inspectors should be organized and diligent in documenting the photos and observations.
2) Blast cleaning can uncover deterioration that was not noticeable during a visual inspection. Additionally, once access platforms are in place during construction, it becomes easier to dig into potential corrosion areas.
3) Keep appropriate contingency allowances during construction. The typical 10% contingency for Class A cost estimates may be too low for steel bridge recoating projects.
4) Maintain a collaborative relationship with the contractor. Many of the repair details for this project were a result of dialogue between the Contractor, the Ministry Representative and the Designer. In some cases, a repair detail from the start of the project was refined / improved mid-project.
5) Anticipate potential repairs and develop repair details that are both flexible in the field (can accommodate different sized repairs) and can be constructed from commonly available materials (plates, angles, bolts). Once it became apparent that significantly more repairs would

be required, galvanized plates were pre-ordered and were cut and drilled on site. Later in the project, regular steel was used and coated with zinc primer before installing as it would also end up being coated with the rest of the bridge.

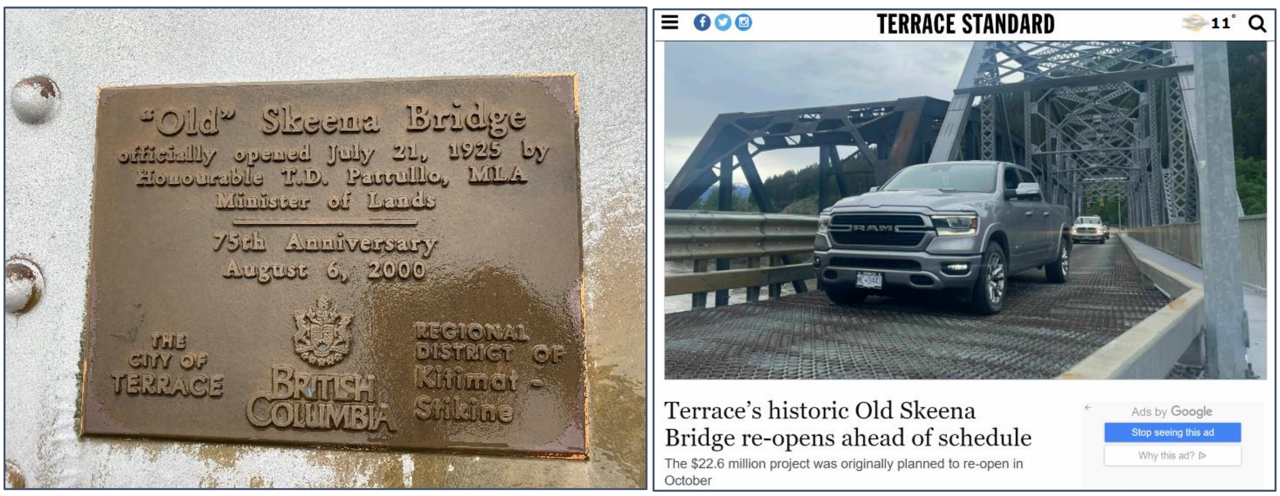
6) Regular coating touchups can extend bridge coating's service life.

Closure

Construction commenced in August 2022, and the bridge was opened to traffic on June 1, 2023 which was three months ahead of the contract completion date of September 1, 2023. With proper maintenance of the coating system, the repair and recoating operations should provide another 30+ years of service for the structure. Additionally, the project achieved improvements to the sidewalk functionality and resulted in a methodology to perform future inspection and maintenance work utilizing a snooper truck.

Success on the project can be attributed to effective and timely collaboration between the Ministry, the contractor and the design team. The Old Skeena River Bridge will achieve 100 years of service on July 25, 2025 (see Figure 27).

Figure 27: 75 Year Plaque from August 2000 (left) and Bridge Opening Headline from Terrace Standard (right)



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