Rising to the Challenge: From Concept through Construction

Repairs of the Existing Taylor Rd. (Callender Hamilton Through Truss Type) Bridge

Grand Falls-Windsor, NL

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Abstract: The Callender Hamilton through truss bridge crosses the Exploits River in Grand Falls-Windsor, NL, and serves as a vital link for both industry and the public in the area. In January 2016, a heavy vehicle struck the bridge's south portal strut and caused severe damage to several non-redundant truss top chord and diagonal members along with several other secondary members. Harbourside Engineering Consultants (HEC) were retained by Newfoundland and Labrador Department of Transportation and Works (NLDTW) to complete a repair design and procedure, with the mandate being to restore the existing structure's full inherent capacity while minimizing design time, bridge closure time, and overall project costs. The project involved a number of challenges requiring an innovative and unique solution. Because the damaged elements included non-redundant members, the loads in these members had to be relieved by introducing an alternate load path prior to their replacement. Due to the site geometry, specifically a near-vertical cliff over 20m high directly in front of both abutments, along with cost and schedule restraints, standard repair methods were conceptualized but ultimately deemed impractical. The solution came in the form of an innovative temporary adjustable-length diagonal jacking strut design, whereby jacking struts were strategically located within the existing truss to create an alternate load path which bypassed the damaged members. A complex jacking system within the struts, including a creative sleeve-type slider system to maintain stability of the strut during the jacking procedure, was developed to maintain bridge geometry and relieve load in the existing damaged members prior to their replacement. A structural evaluation of the bridge superstructure was also part of HEC's scope of work as the live load carrying capacity of the bridge was never verified since its construction in the 1960's. The structural evaluation concluded that a number of structural elements did not meet Canadian Highway Bridge Design Code (CHBDC) CAN/CSA S6-14 requirements. As such, a combination of bolt material testing (to verify the existing bolt strength properties) and posting axle limits were recommended for the bridge following completion of the vehicle collision repairs. In addition to completing the detailed repair design and procedures, HEC provided an on-site Engineer for supervision and direction during all phases of the repair works, including the critical jacking sequences. The project was ultimately a success, being completed safely, on budget, and just marginally over schedule while meeting the main objective: reinstating the inherent load carrying capacity of the structure.

1 Introduction

1.1 Existing Bridge Background Information

The Callender Hamilton Bridge, located in Grand Falls-Windsor, NL, spans the Exploits River (see Figure 1) and serves as the most direct and only maintained access to the vast area of land on the south side of the Exploits River. This land is used for many purposes, both commercial and recreational, and is therefore a vital link for both industry and the public. The bridge is a single lane, galvanized steel through truss bridge with a 76.2 metre (250 foot) span, designed by the English engineering firm Balfour Beatty in the early 1960's and constructed in 1962. The total useable bridge width (i.e. distance between curbs / barriers) is 3.98 metres, while the bridge was designed to provide a maximum vehicle height clearance of approximately 4.68 metres.

The bridge was constructed by a pulp and paper company that operated in Grand Falls-Windsor, but subsequent to the mill closure became part of the NLDTW bridge inventory. Although the general service history of the structure was known to NLDTW, there was no information available in relation to design loading or live load capacity at the time of this project.

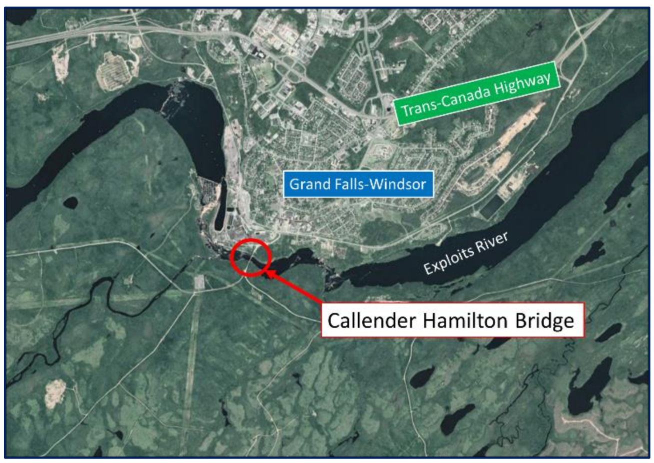

Figure 1: Location of Callender Hamilton Bridge

1.2 Collision Details and Project Scope

In January 2016, a truck carrying a piece of tree harvesting equipment and travelling at a relatively high speed collided with the south portal strut of the bridge. This collision resulted in severe damage to several superstructure members. The most critical members sustaining damage were the non-redundant truss top chord and diagonal members. The Callender Hamilton Bridge following the collision is shown in Figure 2 and Figure 3 below, with damage to the south portal and truss top chord clearly visible in Figures 2 and 3, respectively.

Figure 2: Damaged south portal of Callender Hamilton Bridge

Figure 3: Buckled truss top chord of Callender Hamilton Bridge

Harbourside Engineering Consultants (HEC) were retained by NLDTW to determine / assess the extents of damage by means of a site visit and provide a repair design / procedure as required to reinstate the inherent load carrying capacity of the structure. In addition, HEC were engaged to complete a superstructure evaluation of the bridge in accordance with Section 14 of The Canadian Highway Bridge

Design Code (CHBDC) CAN/CSA S6. Due to the importance of the structure, providing a vital link to the south of the Exploits River, the mandate of the project was to restore the bridge to its full inherent live load carrying capacity while minimizing design and bridge closure times as well as keeping project costs at a minimum.

Upon inspection of the bridge, HEC personnel determined that several non-redundant trusses, along with some secondary bracing elements were damaged beyond repair and required replacement. In addition to the challenges presented by the economic / schedule constraints and the critical nature of the truss members to be replaced, the location of the bridge – supported on two near-vertical cliffs and spanning a large river over 20 metres below – presented its own challenges. This was a unique and challenging project requiring an innovative solution.

2 Key Project Considerations

2.1 Replacing Non-Redundant Structural Members

The structural members damaged during the collision included two truss top chord sections, four truss diagonals, two end portal cross beams including bracing and two top-chord transverse wind bracing members, all of which were damaged beyond practical repair and required replacement. The extents of the damage to the bridge and the members that required replacement are indicated in Figure 4 below.

The truss top chord sections and truss diagonals are non-redundant members, meaning that there are no other members (i.e. no alternate load paths) to share / support the non-redundant member tributary loads. The failure of a non-redundant member would lead to structure collapse. For this reason, the repair methodology needed to provide an alternate load path to the structure to temporarily support the loads currently supported by the damaged truss members to enable their replacement. As can be seen in Figure 4, without any one of the aforementioned damaged truss top chord sections or truss diagonals in place, the structure becomes unstable and there is no path for the vertical loads to be transferred to the south abutment.

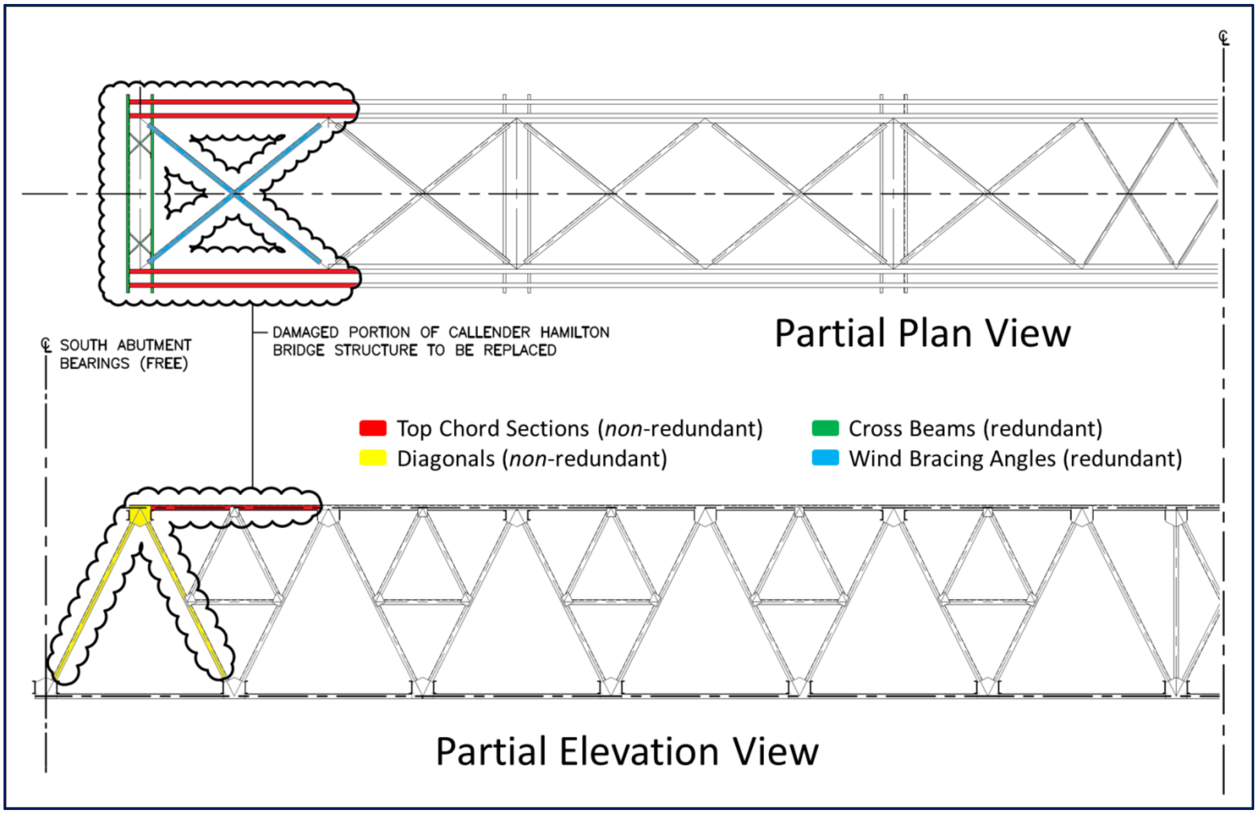

Figure 4: Extents of Callender Hamilton Bridge requiring replacement: partial plan view (top) and partial elevation view (bottom); only south half of bridge shown, north half similar

2.2 Superstructure Vertical Profile

The existing structure was constructed on a constant longitudinal slope. Another significant challenge in the project was to maintain the existing bridge geometry during and following repairs. Without proper procedures and repair methodology, when one relieves the loads (i.e. strains) from the damaged truss members, the members undergo change in lengths and the bridge deforms in a sagging profile. Although the anticipated vertical deflection resulting from the change in strains is relatively small, approximately 13mm, this has a significant effect when considering field fit-up of the new replacement truss members. The 13mm deformation would be visible as a distinct “kink” in the end of the bridge.

2.3 Site Geometry

The location and position of the bridge, with a 76.2m span over the Exploits River whose substructure (i.e. abutments) are supported adjacent to 20m tall and near-vertical cliffs, presented its own challenges to the project. An elevation view of the bridge spanning above the Exploits River can be seen below in Figure 5. A standard method of repair for bridges that have sustained similar damage involves the construction of a temporary steel bent below and adjacent to the damaged portion of the truss in order to support the remainder of the bridge span, relieve the loads in the damaged members, and enable replacement in-kind. However, due to the significant drop below the bridge superstructure and the near-vertical face of the cliff directly in front of the south abutment, it was not reasonably feasible with the project budget and schedule to construct such a bent beneath the existing damaged section of the bridge. As such, several more innovative repair options were conceptualized and considered, all of which are further explained below in Section 3.

Figure 5: Callender Hamilton Bridge spanning the Exploits River (south abutment at right end)

2.4 Budget and Schedule Limitations

The Callender Hamilton Bridge is a vital link between the town of Grand Falls-Windsor and the multi-purpose lands to the south over the Exploits River. Therefore, the structure had to be repaired (i.e. have its inherent load carrying capacity restored) as quickly as possible. It was imperative that the time required for design and bridge closure times be kept to an absolute minimum and NLDTW mandated that HEC's repair design and procedure were detailed such that the Contractor had a reasonable means to complete all works within a one-week full bridge closure.

3 Repair Concepts

3.1 Preliminary Concepts

As previously indicated, a typical repair scheme such as constructing a temporary steel bent beneath the existing bridge to support the remainder of the bridge during in-kind member replacement was not feasible due to site constraints. Taking into consideration the site constraints, HEC first performed preliminary engineering considering a number of possible repair strategies.

The first concept was a partial bridge de-launch. Essentially, this concept involved the following procedure:

- Jack up the bridge superstructure off of the bearings so that the underside of the bridge superstructure would be above the approach roadway;
- Add a truss panel to the north end of the bridge (end of bridge opposite to the damaged end);

- Launch the bridge longitudinally towards the south so that the damaged truss panel is above land with the panel extension added to the north end of the bridge forming part of the bridge span;
- Replace the members in-kind;
- Launch the bridge back into its original position;
- Remove the added north truss panel; and,
- Lower the bridge superstructure back down onto its original bearings.

Although this is a feasible concept in theory, the construction costs and time required to complete the on-site works were considerable and did not meet the project requirements.

The second option considered by HEC was to construct a temporary steel bent on the south approach with two large cantilever beams that would extend longitudinally above the existing bridge top chords and connect to the non-damaged portion of the bridge, in-span of the damaged panel. One would then use the temporary bent to jack up the longitudinal beams / existing bridge and hence relieve the loads in the damaged members. The members could then be replaced in-kind. This concept, although more reasonably feasible than the de-launch option, was costly due to the amount of steel and other temporary works required to construct the steel frame and the large longitudinal beams, and also resulted in significant challenges with respect to the logistics of replacing the members. The cranes used to lift the segments would have to be placed on the approach, behind the temporary frame. In addition, the longitudinal beams, located above the existing top chords, would make it very challenging to get the damaged truss members out and the replacement members in.

3.2 Final Concept

Following development of the above noted concepts, an alternative method that would accommodate the challenging site geometry while still adhering to the budget and schedule limitations was needed. At the conceptual stage, the ultimate solution that met all key project considerations was a temporary diagonal strut solution in which the struts would be located in the plane of the existing trusses connecting the south abutment bearing node to the second to last truss top chord node. This created an alternate load path that effectively bypasses the damaged truss members. The concept is demonstrated in Figure 6 below.

There were three major challenges associated with bringing the temporary strut concept to reality, which are further described as follows:

3.2.1 Geometric Constraints

The first major challenge involved physically laying out the temporary struts. Struts could not simply be placed in the same plane as the existing damaged truss members, since the existing members would be in the way and they could not be removed until the temporary support system was fully installed. Therefore, to successfully implement the temporary diagonal strut concept, the struts would need to be placed in the same plane as the existing trusses while keeping the damaged truss members in place.

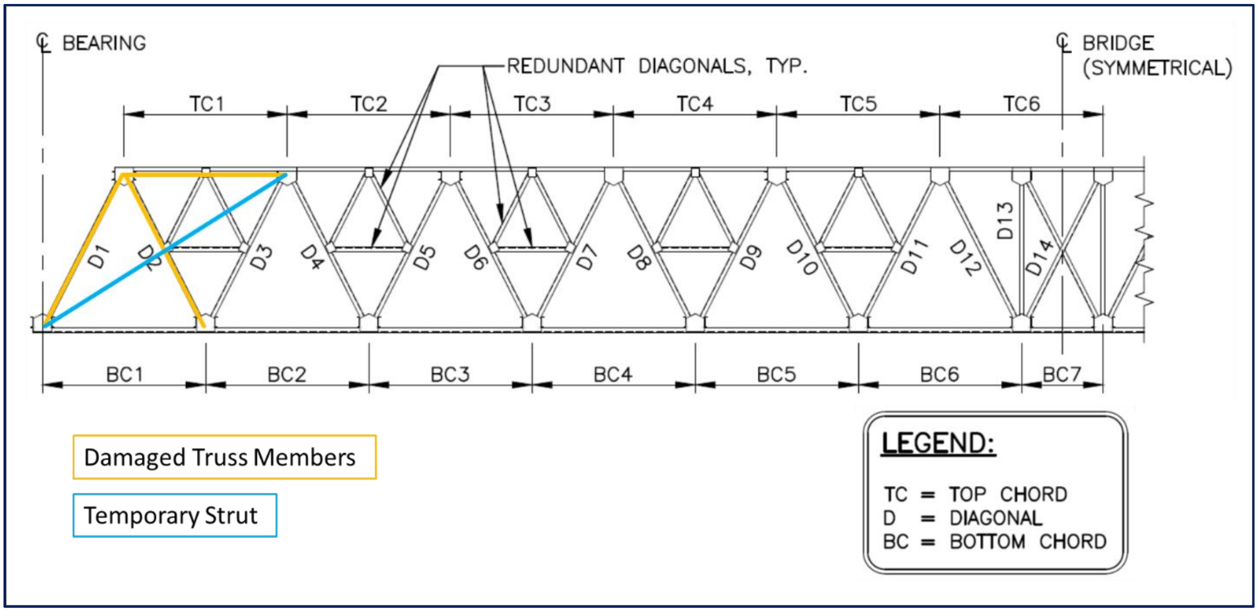

Figure 6: Temporary support concept, in which a temporary strut provides an alternate load path to allow the replacement of the damaged truss members

The challenge of placing the struts in line with the existing truss was solved by using built-up steel struts, each comprising of two channel sections tied together using batten plates. The strut components were assembled in the field and literally constructed around the existing truss diagonals with the diagonals passing through openings between battens in the fully assembled struts. Other challenges faced with the strut geometry were positively connecting the struts to the existing bridge gussets and avoiding the existing bridge barriers, both which were overcome with intricate steel detailing.

3.2.2 Load Transfer / Bridge Vertical Profile

The second major challenge concerned how loading would be transferred from the existing damaged members into the temporary struts and vice versa following member replacement. In order to maintain the structure geometry and mitigate dynamic loads, we needed to develop a method to release loads in the existing members and introduce loads to the temporary struts in a controlled manner.

Gradual loading of the struts while maintaining the bridge geometry was accomplished through the design of an innovative adjustable length jacking strut system. Essentially, the struts were composed of two separate segments that were joined using an adjustable sleeve (pipe-in-pipe) section. The adjustable sleeve enabled the jacking of the struts while in place to relieve the loads in the existing truss members and maintain bridge geometry.

An alternate load path also needed to be provided to get the transverse wind loading from the plane of the top chords down into the south abutment bearings during the member replacement. The solution involved the incorporation of a horizontal strut, connecting the top of the two built-up diagonal jacking struts, along with two cross bracing bars (wind bracing) tying the horizontal strut down to the bottom of the jacking struts near the abutment bearings. The wind bracing was required since the members being replaced were port of the bridge end portal frame.

3.2.3 Existing Bridge Analysis for Alternate Load Path

The third major challenge was to ensure that the existing bridge elements could resist the loads resulting from the change in load path (i.e. while the load was in the jacking struts during the member replacement). The temporary diagonal jacking strut, spanning from the south bearing node to the second to last top chord node, is orientated flatter than the standard bridge diagonals. Since diagonals are the main vertical load carrying elements on the bridge, one can clearly see based on trigonometry that for the same vertical load, the axial loads in an 'axial only' member will increase as its orientation away from vertical increases. As a result of the diagonal struts being orientated in a flatter position than the existing diagonals, the temporary diagonal struts introduced longitudinal loads in the truss nodes and end bottom chord members that significantly exceeded the existing bridge design loads.

A detailed analysis of all existing bridge components affected by the proposed change in load path resulting from the diagonal strut installation was undertaken. The analysis concluded that strengthening was required on the temporary strut connection nodes (i.e. south bearing nodes and second to last top chord nodes). As can be seen below in Figure 8, strengthening angles were designed and implemented to accommodate the higher temporary jacking strut loads

4 Rehabilitation Methodology

Understanding the complexity of the repair design and procedure, NLDTW engaged HEC to have a Field Engineer on-site during all phases of construction to perform inspection and also to provide guidance to the Contractor as required.

The rehabilitation of the Callender Hamilton Bridge consisted of the five following phases:

1. Installation of the temporary jacking struts, horizontal struts and wind bracing.
2. Jacking of the struts to relieve the load in the existing damaged members.
3. Replacement of the damaged members.
4. Transfer of loads into the newly installed permanent truss members.
5. Dismantling and removal of the jacking struts and other temporary works components.

Due to the required mobilization of contractor equipment, the Callender Hamilton Bridge was fully closed to traffic for the extents of the repair procedure to minimize the overall length of construction and ensure overall public safety.

The first step to be taken in constructing the temporary jacking strut system involved the installation of temporary bracing plates to provide lateral stability to the top chord members prior to making any alterations to the structure. The individual components (i.e. angles) of the built-up truss top chords are tied together by intermittent bolts and batten plates as shown on the left of Figure *7* below. Global stability of the truss top chords is provided by braced points: vertically by gusset plates at the truss diagonal / redundant diagonal nodes and horizontally by connector plates at truss diagonal nodes as shown on the right in Figure 7 below.

Figure 7: Batten plates *(left)* and connector plates *(right)* providing stability to the top chord

In order to attach the temporary jacking struts to the top chord nodes, each strut was bolted to a connector plate (see Figure 7 above). The connector plates, as mentioned above, provide transverse stability to the truss by tying the vertical gussets together. In order to permit the attachment of the jacking strut to the connector plate, the bolts had to be temporarily removed from the connector plate so that the end plate of the strut could be bolted to the connector plate. To avoid temporary reduction in lateral stability of the top chords during the installation of the jacking strut end connection, bracing plates were installed on the top surface of the top chords directly above the connector plates. An installed temporary bracing plate is shown on the left in Figure 8 below.

Once the temporary bracing plates were installed, the built-up temporary jacking struts were assembled around the existing truss diagonals. Each jacking strut was composed of two MC310 structural steel members interconnected at several locations along their lengths via bolted batten plates, strategically located to avoid conflict with the existing truss elements. The jacking system used to load the jacking struts was located near the lower node of the temporary struts for worker accessibility purposes. Each jacking system consisted of two different diameter circular HSS (hollow structural section) pipes centered on the strut centerline. Ring plates were welded around the outside surface of the smaller HSS, ensuring the gap between the two pipes was very small to minimize potential lateral displacement between the two segments. The innovative sleeve systems played a key role in the jacking system as they provided lateral stability to the system in all directions during the jacking procedure while still allowing the length of the strut to be adjusted. A jack was placed on either side of the HSS sleeve system (i.e. two jacks per strut) and would eventually be used to apply load to the jacking struts and relieve the load in the damaged members. Four threaded bars with nuts on either end were also included into the design, along with shim plates to "lock" the jacking strut lengths in place after the jacking procedure so that the jacks were not relied upon to support the bridge dead loads during replacement of the truss members. In addition to the main jacking struts, temporary braces and strengthening angles were installed to provide additional support to the system. A fully assembled jacking strut is shown in Figure 8. It should be noted that the jacks were not loaded at the stage of the procedure.

Figure 8: Temporary bracing plate *(left)*; jacking strut assembled around existing truss members *(right)*

Before commencing the jacking procedure, the top strut and threaded bar cross bracing were installed between the two jacking struts to provide resistance to lateral wind loads. It should be noted that the threaded bar cross bracing was installed slack in this condition, so that they would not restrict (or take on loads) during the jacking process (which effectively elongates the temporary support system). The temporary support system is shown schematically in Figure 9 and Figure 10.

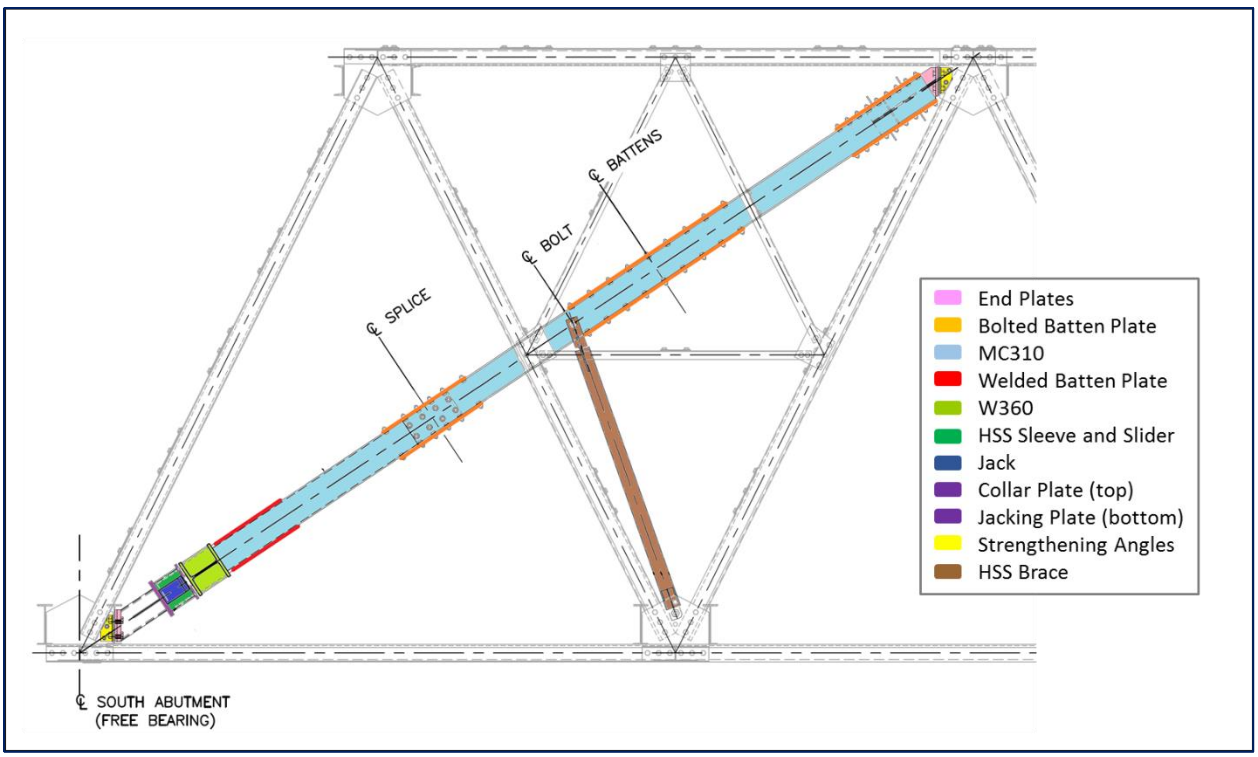

Figure 9: Temporary built-up jacking strut and jacking system *(elevation view)*

Prior to proceeding with the jacking, the jacks were all calibrated and the pair of jacks for each strut were equipped with a jacking manifold, which ensured both jacks were supplied with and maintained equal pressure during the jacking, thereby ensuring that the jacking struts were loaded and lifted concentrically without inducing unwanted moments into the struts. Prior to pressurizing the jacks, the threaded bars were loosened to ensure they were not taking any load during the jacking procedure, hence enabling the strut to elongate along its longitudinal axis. The jacks were then slowly pressurized until contact was made between the jack plunger and the underside of the W360 and the total stroke of the jack was recorded for reference. It should be noted that HEC completed detailed calculations to determine the theoretical load required to relieve the loads in the existing damaged truss members to be replaced and the jacking procedure was developed based on achieving this target load in the jacks. However, there are many factors in the field, such as; temperature, jack calibration, bolt slip, and differences between actual and predicted material weights, all of which can affect the results during jacking. Therefore, as a verification above and beyond the jacking loads, HEC also determined the predicted jacking stroke required to obtain the target jacking loads. The predicted jacking stroke did not only account for change in strain in the existing members during the relieving of loads but also included the elastic shortening of the jacking struts that occurs during jacking. Therefore, once the reference stroke was determined, the jacks were pressurized to approximately 25% of the target load, locked off, and the change in stroke from the reference stroke was recorded and compared to the predicted theoretical stroke. This process was repeated until the jacking reached 100% of the target loads, which was equivalent to approximately 450 kilonewtons (kN) per jack. The observed movements (i.e. total jacking strokes) were slightly larger than the theoretical estimated values, which HEC contributed to bolt slip that occurred during the jacking procedure.

Once it had been confirmed that an adequate amount of load had been transferred to the jacking struts, the jacks were locked off, shim plates were installed between the two strut segments (W360 bottom flange and collar plate) to provide longitudinal continuity to the strut, and the threaded bars were snug tightened. The placement of the shim plates and tightening of the threaded bars served to secure the jacking struts in their extended positions. The pressure was then then slowly released from the jacks, gradually transferring the loads from the jacks to the shim pack / internally into the struts. The cross bracing threaded bars were then tightened into a snug, strain-compatible condition to provide a load path for transverse wind loads during member replacement. At this stage, all loads were relieved in the existing truss members, all temporary works assemblies were installed, and the Contractor was ready for member replacement. A fully-constructed and fully-loaded built-up jacking strut / jacking system, with all components labelled, is shown in Figure 11.

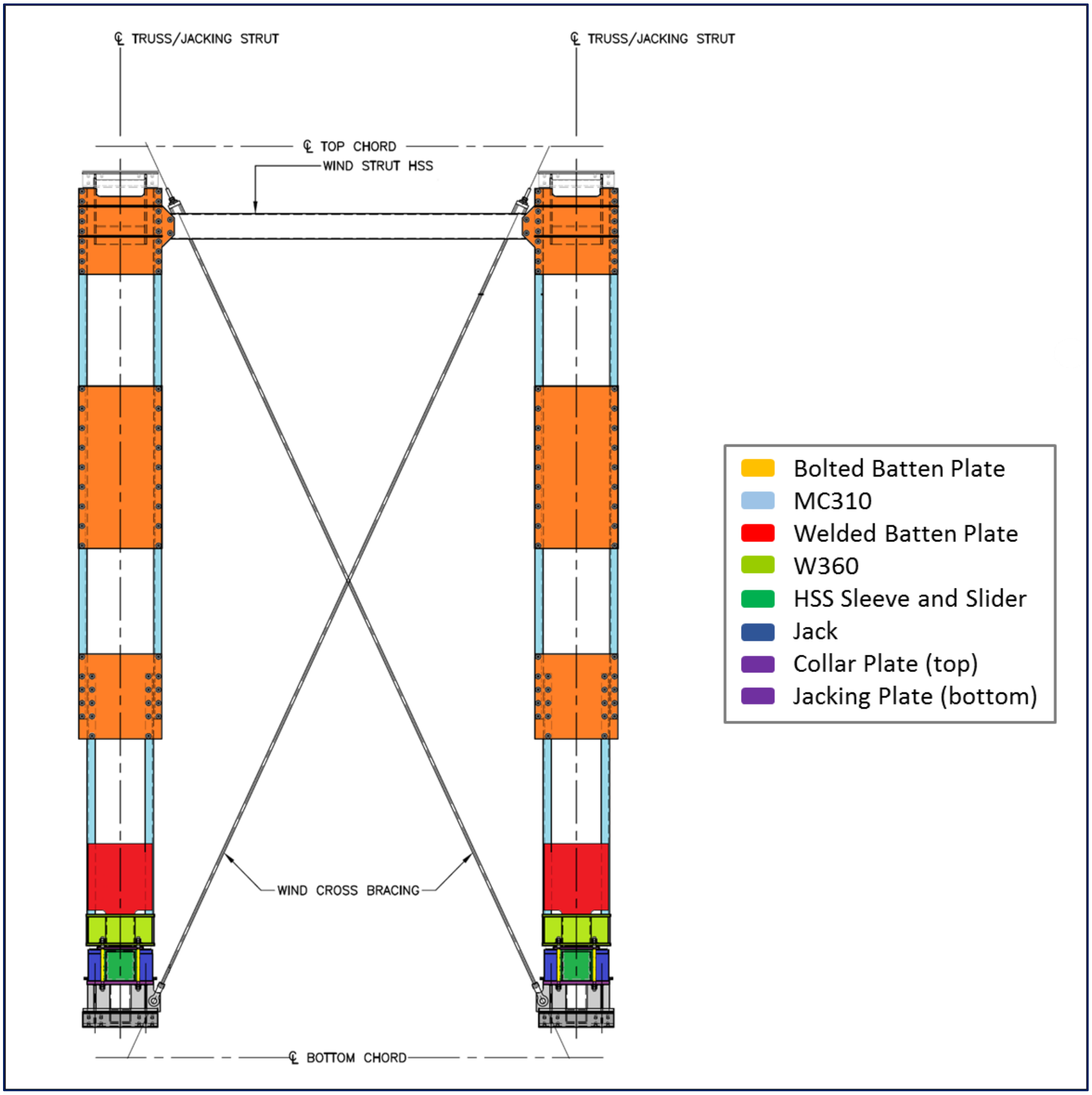

Figure 10: Temporary built-up jacking strut and jacking system *(front view)*

Once the jacking struts had been secured and the jacks depressurized, the second-to-last diagonal in each truss (D2 in Figure 6 and Figure 14) were carefully torch cut. Although the jacking procedure was designed to theoretically relieve all loads in the truss members to be replaced, there are uncontrollable variables that generally result in a small amount of residual load remaining in the truss members. Therefore, any residual load remaining in the truss members requiring replacement was dissipated when the D2 diagonals were torch cut. The D2 diagonals were selected for torch cutting because they are tension members while the end diagonals and truss top chords, D1 and TC1 respectively, are both compression members. Cutting a tension member results in a rebound away from the torch cut, which is safer for the construction workers than torch cutting a compression member, which rebounds towards the torch cut location under load.

With all damaged truss members completely relieved of load following the torch cutting of the D2 diagonals, the contractor was able to proceed with the damaged member replacement. All damaged members, as indicated in Figure *4* above, were removed and replaced in-kind. Man lifts, specifically a scissor lift and a telescopic boom, along with two mobile cranes were used by the Contractor during site works as shown in Figure 12 below. The contractor also had a crane supported man basket to perform work outboard of the trusses. All newly installed members were inspected by HEC personnel, ensuring all components and connection plates had been installed and all bolts were fully pre-tensioned using the turn-of-nut method.

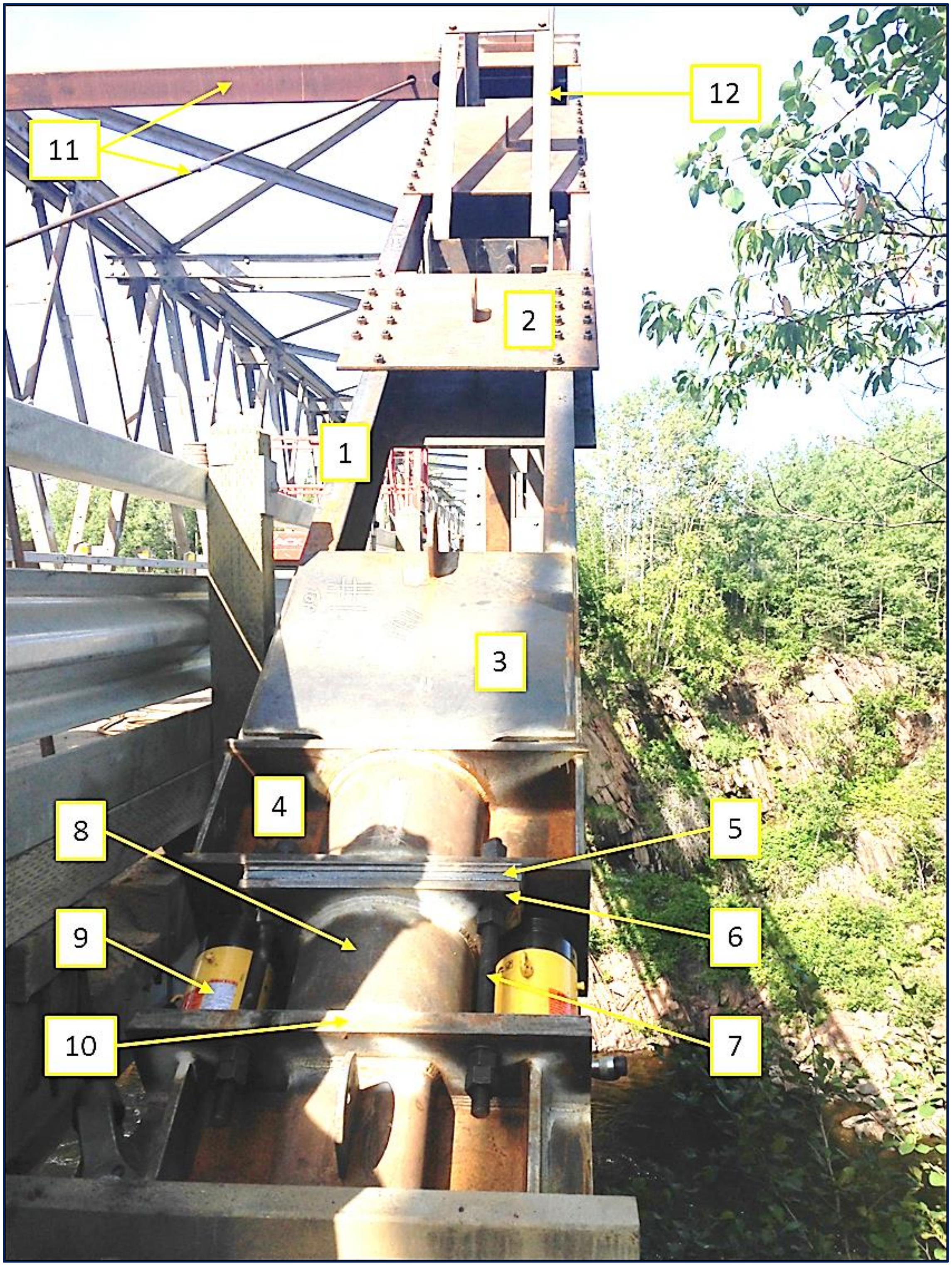

Figure 11: Assembled and loaded jacking strut and jacking system *[1. MC310, 2. bolted batten plate, 3. welded batten plate, 4. strengthened W360, 5. shim plates, 6. collar plate, 7. threaded bar, 8. HSS sleeve, 9. jack, 10. jacking plate, 11. wind strut and wind cross bracing, 12. existing truss diagonal]*

Following the installation and inspection of all new members, the load had to be transferred from the temporary jacking struts into the newly installed truss members using the same set-up procedure as explained above for the initial jacking. Prior to initiating the jacking, the threaded bars were loosened so to not restrict the jacking operation. The jacks were then slowly pressurized to 500 kN per jack (1000 kN per truss) to ensure all loading was removed from the shim plates, therefore relieving load from the middle of the struts. The shim plates were carefully removed and then the pressure in the jacks was slowly released, thereby transferring all load from the jacking struts into the newly installed bridge elements.

With the newly installed truss members carrying all loads, the temporary support system was disassembled and removed. The temporary wind cross bracing and the wind strut were the first to be removed, followed by both jacking struts including their jacking systems and all temporary braces and strengthening angles. All holes that were drilled in existing plates for connection of temporary works were filled with fully pre-tensioned, matching diameter, galvanized bolts for corrosion protection. In addition, all damage to existing or newly installed truss members was touched up with two coats of zinc-rich paint, also to provide corrosion protection.

Figure 12: Truss member replacement operations underway *(left)*; newly installed truss members *(right)*

5 Structural Evaluation

5.1 Evaluation Findings

Aside from the collision damage repair design and procedures, HEC conducted a full superstructure evaluation, in accordance with Section 14 of the CHBDC, to determine the live load carrying capacity of the existing bridge. During this evaluation, it was determined that a number of structural elements did not meet the CAN/CSA S6-14 (CHBDC) requirements for the full CL-625 live loading, the current design truck used for the design of highway bridges in Canada. When the bridge was designed and constructed in the early 1960's, a lighter HS20-44 design truck was the standard live loading considered when designing highway bridges in Canada. It should be noted that the bridge does not appear to have been under-designed, although some of the structural elements were efficiently designed according to the HS20-44 design truck, which as a result of the increased live loading does not meet the current requirements the CHBDC. The structural members that did not meet CHBDC CL-625 loading requirements included the timber decking, the longitudinal stringers, below deck crossbearers, and four truss diagonals (one diagonal, mirrored at all four truss corners). The aforementioned overstressed members have been highlighted for reference in Figure 13 below. It should be noted that the longitudinal stringers and truss diagonals were only marginally overstressed (i.e. less than 5%), while the timber decking and cross bearers were more significantly overstressed, with the crossbearers governing at 35% overstress. As a comparison to the current CHBDC CL-625 truck, HEC calculated the demands in the crossbearers using the historical HS20-44 design truck and determined that the overstress was only 3% (i.e. in other words a reasonable and efficient design). The main reason for the increase in overstress in the crossbearers (which are local components governed by axle loads, rather than governed by full bridge global loads) is the fact that the axle spacing for the CL-625 truck is significantly closer than the axle spacing on the HS20-44 truck and as a result the crossbearers are required to resist nearly two full 125 kN axles of the CL-625 truck at spacing of 1.2 meters vs rather than just the one 142 kN axle of the HS20-44 truck.

HEC calculated vehicle weight restrictions for the bridge, governed by the crossbearers, based on Section 14 of the CHBDC, but noted that the calculated restrictions appeared to be overly conservative. Further investigation by HEC indicated that the CHBDC does not seem to suitably differentiate between local component overstresses and global overstresses and the restrictions calculated per the CHBDC would be overly conservative from a global truck weight perspective (i.e. limiting the overall truck weight for no reason). HEC therefore completed an independent analysis to compare the maximum allowable unfactored axle loads based on full crossbearer utilization (using load factors and member resistances in the CHBDC) to the NL legal axle and axle group weight limits. Refer to Table 1 below for axle comparisons. As can be seen, only the legal tridem axle weight limits cause an overstress on the crossbearers.

Figure 13: Structural members not meeting CAN/CSA S6-14 requirements to carry full CL-625 loads: timber deck (blue), truss diagonals (green – fifth member from each end, both trusses), longitudinal stringers (yellow, below deck), and crossbearers (red, below deck)

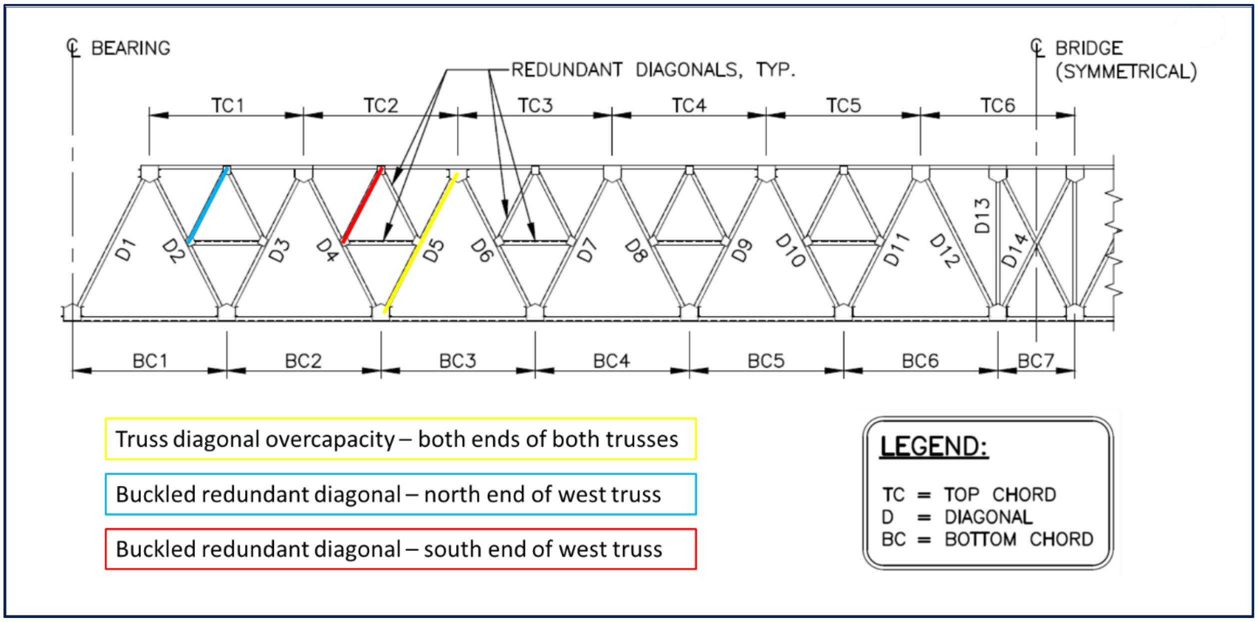

Figure 14: Locations of overcapacity and buckled diagonal truss members (one side of one truss shown)

Table 1: Allowable Unfactored Axial Loads (compared to actual NL Legal Weight Limits)

AXLE TYPE	α_L	DLA	AXLE SPACING	MAX ALLOWABLE UNFACTORED LOAD	NL LEGAL WEIGHT LIMIT
Single	1.49	0.4	N/A	158 kN (16.0 Tonnes)	9.1 Tonnes
Tandem	1.49	0.3	1.2m	179 kN (18.3 Tonnes)	18 Tonnes
Tridem	1.49	0.3	1.2m	201 kN (20.5 Tonnes)	21 Tonnes
Tridem	1.49	0.3	1.5m	218 kN (22.2 Tonnes)	24 Tonnes
Tridem	1.49	0.3	1.8m	237 kN (24.2 Tonnes)	26 Tonnes
Quad	1.49	0.3	1.2m	234 kN (23.9 Tonnes)	N/A

The member connections and structural bolts used in the connections throughout the bridge were also analysed as part of the superstructure evaluation. The original contract structural drawings made available to HEC for the structural evaluation did not present any information relating to the bolt material properties, except for stating that the bolts were either high strength (HS) or mild steel (MS). All bolts in the trusses were graded HS and it was apparent, based on the steel strengths indicated on the drawing, that all of the steel for the bridge was fabricated and specified according to European Standards. As such, and since the strength of the bolts could not be verified, bolt strengths were assumed based on allowable stresses presented in the Historical Structural Steelwork Handbook (*written by W. Bates and published by the British Constructional Steelwork Association Ltd., April 1991*): 108 Megapascals (MPa) for HS bolts and 77 MPa for MS bolts. These values appeared to be excessively conservative, but in the absence of alternate sources of information, were used in the superstructure evaluation. HEC's analysis, as predicted due to the low allowable stresses, indicated that end connection bolts in 12 truss diagonals (144 bolts total) were theoretically overstressed for CL-625 live loads as per the CHBDC.

Additionally, it was observed during HEC's site visit that two redundant diagonals were buckled (likely due to previous vehicular collisions). The redundant diagonals in the structure do not carry any load, but play an important role in the stability of the structure as they serve as braces for the non-redundant diagonals and top chord members to reduce their effective bucking lengths. The locations of the buckled diagonal truss members are indicated in Figure 14 above.

5.2 Evaluation Recommendations

Strengthening and/or replacement of the crossbearers not meeting CHBDC requirements was deemed impractical due to the number of overstressed crossbearers (26 in total) and the challenges associated with difficult access for site works below the bridge deck, which would lead to significant costs. Based on HEC's additional analysis, where the crossbearer capacities were compared to the NL legal axle weight limits, HEC recommended posting an axle limit sign based on the actual capacity of the crossbearers (as shown in Table 1 above). This recommendation results in no weight restrictions when compared with NL legal load limits for single and tandem axles and only an 8% maximum restriction on tridem axles.

HEC recommended two options to NLDTW with respect to the overstressed bolts; replace the bolts one at a time in the field with new ASTM A325 bolts, or complete material testing on the bolts to determine their actual yield and tensile strengths (as it was believed that the values provided in the Historical Structural Steelwork Handbook were over-conservative).

With respect to the bolts, the first course of action selected by NLDTW was to replace the bolts one-at-a-time since the Contractor was already on site to complete the vehicle collision repairs and the bolts could be replaced at little cost. However, this proved to be more challenging than originally anticipated as it was apparent when trying to replace the bolts that they were not designed to be slip critical. As such, the bolts in the field were resisting shear forces in bearing and were very difficult to remove under loading. After obtaining this feedback from the Contractor, NLDTW decided to proceed with the materials testing and 15 existing truss bolts that were removed during the repair procedure were sent to Atlantic Metallurgical Consulting Limited (AMC) in Dartmouth, NS, for materials testing. Based on the results of AMC's tests, the samples bolts were found to have an average tensile strength of 609 MPa. Due to the relatively small sample size tested, HEC calculated a coefficient of variance to assess the reliability of the tested average tensile strength. The tensile strength ultimately used for evaluation of the structure, based on a procedure presented in the CHBDC, was 554 MPa. As anticipated, the material testing of the bolts resulted in a considerable increase in the calculated bolt capacity and bolt replacement in the truss diagonals was no longer necessary since the revised bolt capacities were sufficient for the connections to meet CHBDC requirements.

Due to the importance of the redundant diagonals for the overall stability of the trusses, NLDTW proceeded with the replacement of the two buckled redundant diagonals as recommended. The replacement procedure, which was designed by HEC and included the installation of a temporary brace to stabilize the truss members during the redundant diagonal replacement, was completed by the Contractor immediately following the vehicle collision repairs.

6 Conclusion

The rehabilitation of the Callender Hamilton Bridge was a very unique and challenging project that required an innovative solution. The bridge is a very important structure to the community of Grand

Falls-Windsor, NL, providing the most direct and only maintained access to the lands south of the Exploits River. The structure therefore had to be repaired as quickly as possible with minimal traffic interruptions. The Steel Fabricator and General Contractor, who completed all site works for the repairs, was Land and Sea Welding Limited from Carbonear, NL. In the end, the repair procedure was completed during a 10 day full bridge closure with the Contractor working 12-hour days. Overall, the repairs project was successful with the on-site work being completed safely, on budget, and only slightly over schedule, while the full inherent capacity of the existing superstructure has now been restored.

The superstructure evaluation also gave NLDTW a better understanding of the live load carrying capacity of their structure. Along with recommendations to ensure the bridge meets the CHBDC requirements, HEC's superstructure evaluation report also included summary tables indicating utilization ratios for all superstructure elements critical to the vertical load carrying capacity of the bridge. This evaluation will prove very beneficial to the Department any time they have to process requests for overloading permits on the bridge.

The project was completed in August 2016 and was ultimately a success, providing an excellent case study that demonstrates that any engineering challenge can be overcome with creativity and ingenuity.