REHABILITATION WORKS FOR PINAWA BRIDGE OVER WINNIPEG RIVER

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Abstract

The Pinawa Bridge over Winnipeg River near Pinawa, Manitoba, built in 1961, is a vital link to the community of Pinawa, the Manitoba Hydro Pointe du Bois Generating Station, and AECL's Whiteshell Laboratories. The structure accommodates two traffic lanes on four 42.7m main spans and two 30.5m approach spans. The bridge superstructure comprising of five steel plate I-girders and reinforced concrete deck, is continuous over the piers with hinges in spans 3 and 5. Finger plate expansion joints above the hinges accelerated deterioration of the underlying deck soffit, steel girders, and pin-hanger system. Rehabilitation works were required to extend the remaining service life due to deterioration of several key components. The major rehabilitation work items included:

1. Replacing existing pin-hanger connections with girder continuity splices to make the bridge fully continuous;
2. Replacing existing rocker bearings with pot bearings to accommodate the change in behavior of the superstructure;
3. Constructing a partial depth deck replacement, widened to current geometric design standard;
4. Installing girder cover plates to increase the design live load and to enable deck widening; and
5. Associated road works to accommodate widened bridge geometry.

The project presented challenges to Manitoba Infrastructure and Transportation (MIT) and to the design team in ensuring that the rehabilitation works occurred in accordance with the design and staging requirements. Challenges encountered during construction works included:

1. Staging of construction works to maintain at least one traffic lane at all times;
2. Installing continuity splices in areas of severely corroded and/or distorted girder steel;
3. Reviewing contractor's staging techniques to allow concurrent continuity splice installation and deck demolition works; and
4. Completing staged partial depth deck construction in Manitoba winter conditions and constructing the widened deck section profile.

The rehabilitation works were completed successfully and will increase the bridge service life by 40 years and meet current design standards.

1 INTRODUCTION AND BACKGROUND

The existing Pinawa Bridge was constructed in 1961. This bridge serves as a link between PTH 11 and the town of Pinawa. This eastern Manitoban town is a recreation paradise offering sailing, canoeing, fishing, hiking, swimming, snowmobiling, and cross- country skiing. The bridge also serves as an important link to Atomic Energy of Canada Limited's (AECL) Whiteshell Laboratories nuclear research facility as well as Manitoba Hydro's Pointe du Bois hydroelectric dam

1.1 Description of Existing Bridge Structure

The structure accommodates traffic in two directions, each with one traffic lane and consists of four 42.7m main spans and two 30.5m approach spans. The clear roadway width on the bridge was 8.53m. Concrete curbs with aluminum traffic guardrails were provided along the entire length of the bridge. The superstructure consisted of five steel plate I-girders supporting a 190mm thick composite, reinforced concrete deck and 38mm asphalt overlay. The bridge superstructure was continuous over the piers with pin-hanger system hinges in Spans 3 and 5, as shown in Figure 1. The bridge superstructure had rocker bearings at each abutment and at Pier Nos. 2 and 4. The bearings at Pier Nos. 1, 3, and 5 were fixed. Finger and sliding plate expansion joints were located at both the hinge and abutment locations, respectively. The substructure consists of two abutments with structural approach slabs and five piers. Both abutments are supported on steel H piles (driven to refusal), whereas the piers are supported on spread footings anchored to bedrock.

The bridge was originally designed in accordance with the standard specifications for highway bridges of the American Association of State Highway Officials, 1957, for H20-S16 and H41-S56 (AECL truck) truck loading.

Figure 1 – View of the pre-rehabilitated bridge (left) and the pin-hanger system (right)

1.2 Detailed Condition Survey

A Detailed Concrete Condition Survey (DCCS) was administered by MIT and completed by National Testing Ltd. / Jacques Whitford Ltd. in October 2008. For detailed condition survey and structural inspection practices, MIT follows the Ontario Structure Rehabilitation Manual (OSRM) and the Ontario Structures Inspection Manual (OSIM), along with MIT's own specific provisions. The DCCS found cracking in the bituminous wearing surface, concrete deck, deck soffit, curbs, and abutments. The condition of the deck waterproofing was observed to be poor and it had also debonded from the deck. The average concrete cover of the deck was measured to be about 42mm which was below currently acceptable standards and corrosion of reinforcing bars was evident. Acid soluble chloride content at the level of deck reinforcing ranged from 0.004% to 0.227% by mass of concrete. The threshold value to initiate corrosion is 0.025% by mass of concrete. Water soluble chloride content was measured from 0.001% to 0.276% at the level of deck reinforcing. The corrosion potential measurements ranged between -89 mV to -500 mV with an average of -261 mV. Approximately 1,300 m2 (65%) of the deck area had corrosion potential between -200 mV and -350 mV. A total of 290 m2 (15%) of the deck area has corrosion potential more negative than -350 mV. The water soluble chloride content tests as well as corrosion potential measurements indicated that chloride had penetrated to the average depth of reinforcing bar and was causing corrosion of the reinforcing steel.

In 2009, MIT completed a detailed structural steel survey (DSSS). The DSSS found the girders displayed varying degrees of coating failure and corrosion of members was also observed, more prominently at the girder/hinge connections located east of Piers Nos. 2 and 4. The severity of corrosion varied from being severe to very severe directly under the joint. Ultrasonic measurements as well as caliper measurements indicated that section losses had occurred on the web as well as the flange plates. Typical section losses were between 10% and 20%. Steel coupon testing from the girders confirmed that all the samples either met or exceeded the requirements of A373-58T standard. The Charpy V-Notch impact toughness tests performed on the girder samples met the requirements of CSA G40.21-04 Category 3 and AASHTO M 270M, Grades 250, 345 and 345W, Temperature Zone 3. The test samples were also analyzed for yield and ultimate strength as well as chemical composition to confirm the steel properties for load rating. Measurements of pin diameter were taken as an initial review of the condition of the pin barrel to determine if any obvious wearing was present and if further investigation was required.

The curbs and finger plate joints were also observed to be in poor condition. The finger plate joints had varying degrees of corrosion. The open joint system had resulted in deterioration of the deck soffit. The abutment bearings showed heavy accumulation of pack rust and corrosion, which was limiting or preventing the bearing from freedom of movement. The abutments and piers were generally observed to be in good condition.

1.3 Load Rating of the Bridge Structure

A load rating evaluation was performed by MIT in accordance with AASHTO LRFD Bridge Design Specifications (AASHTO) and Manual for Bridge Evaluation (MBE). The load rating analysis showed that the girders could sustain all the legal loads at the inventory and operating levels. However, the girders were evaluated to be inadequate in sustaining the design truck HL-93 at the inventory levels and HSS 25 and HSS 30 design truck loads at both inventory and operating levels. The load rating factors for HSS 30 were below 1.0 at different sections of the girders. Therefore, it was recommended that the girders be strengthened by installing cover plates at the identified sections (Dorton, A. 1995).

1.4 Rehabilitation Needs

Based on the structure's age and observed deterioration, the Pinawa Bridge and adjacent roadways required extensive rehabilitation. The rehabilitation works included:

1. Repair of existing deteriorated structural steel caused primarily by corrosion;
2. Reconstruction of bridge deck and barrier;
3. Upgrading of roadside safety details on the bridge to meet current MIT design standards; and
4. Strengthening of the bridge to accommodate increased bridge design loads.

These rehabilitation works were required to extend the lifespan of the bridge for an additional 25 to 40 years while adequately meeting the needs of all the bridge users in a cost-effective manner. This was required to be accomplished by widening the deck overhangs as much as practically possible without adding new girder line(s) and widening the substructure. Tetra Tech was retained by MIT to complete the preliminary design, detailed design, and Advisory Services During Construction (ASDC) for the rehabilitation works. MIT completed the contract administration and construction inspection for the project.

2 PROJECT SCOPE AND METHODOLOGY

Based on the rehabilitation needs and site constraints, the engineering services were developed and delivered in three discrete stages:

1. Stage 1
    a. Condition assessment of the bridge structure;
    b. Conceptual design of bridge deck widening options;
    c. Development of the selected bridge widening option including implementing geometric improvements to the structure;
    d. Presentation of the chosen option to the public for information purposes and to review how the proposed rehabilitation works would impact the impacted stakeholders;

2. Stage 2
    e. Detailed design of rehabilitation works;
    f. Preparing tender drawings and specifications;

3. Stage 3
    g. ASDC during rehabilitation works; and
    h. Contract administration and construction inspection

3 REHABILITATION FACTORS INFLUENCING REHABILITATION STRATEGY

In addition to addressing the rehabilitation needs based on the observed deterioration and increasing structural capacity to operate at current design levels, it was paramount to MIT as part of the project that the rehabilitation works implemented details to either eliminate or mitigate future maintenance requirements that could potentially impact roadway users. Based on the age, maintenance / repair history of the structure, and structural details, MIT chose to implement conservative yet practical design and construction details which would minimize risks for completing proposed works on the structure.

3.1 Geometric Design Criteria and Safety

MIT's Highway Planning and Design Branch recommended a minimum 9.6m roadway width based on the Geometric Design Criteria (GDC). This recommendation necessitated increasing the existing bridge width.

The existing railing over the bridge consisted of concrete curbs and aluminum posts and railings. It was noted during the DCCS that the concrete covering the rail post inserts on several of the post locations had spalled off due to inadequate concrete cover. The existing approach guardrail on the roadway approaches to the bridge was a w-beam / timber post system. For the approach guardrail and bridge barrier design, consideration was given to the incidence of accidents, future speed and volume of traffic, geometrics of the highway at the structure, conformance with crash testing standards and maintenance requirements due to barrier impact and snow clearing operations when evaluating whether to rehabilitate or replace the existing bridge barriers, bridge curbs, and approach guardrails.

Based on discussions with the project stakeholders, that included MIT's roadway and bridge maintenance section, MIT's traffic safety section, and regional roadway design section, and considering the age of the structure, and the required increase in protection level to AASHTO crash test level 5 (TL-5), it was decided to replace the existing barriers and curbs with an F-shaped barrier and to provide a lengthened guardrail that meets current design standards. A TL-5 concrete barrier was chosen for this bridge for the desired protection level and in order to maximize the clear width of the bridge deck.

3.2 Hinge and Pin Replacement

The girder steel at and adjacent to the existing pin hanger connections was observed to have areas of very severe deterioration during the DSSS due to deck joint leakage. Although not specifically observed during the DSSS, corrosion at pin hanger connection can have several detrimental effects on the pin which can potentially lead to catastrophic failure of the connection, which may include:

1. The cross-section of the pin can decrease due to corrosive section loss. This corrosion can produce pitting that may act as crack-initiation sites.
2. Corrosion can effectively lock the pin within the connection so that no rotation about the pin is permitted. This can lead to large torsional stresses within a reduced section of the pin. The torsional stresses, combined with the shear stresses, provide a likely location for the development and propagation of cracks and the eventual failure of the pin.
3. Crevice corrosion (pack rust) can develop between the hinge link plate causing the pin to migrate out of the connections, causing failure of the superstructure at the connection

No ultrasonic inspection was performed on the hanger pins and additionally, locating cracks that initiate on the pin barrel at the shear plane perimeter was a difficult task. The shear plane is not visible unless the pin is removed from the connection - a labor- and equipment-intensive task. Although, presence of any cracks or loss in cross-section of the pin was not observed as well as rotation about the pin was still permitted, however, based on the age of the structure and the presence of very severe section losses of the girder steel adjacent to the hanger, it was recommended to either replace the pins or replace the pin-hanger system with new bolted continuity splice connections. In addition, the option of installing catcher beams under the hinges required consideration as it would provide more redundancy to the pin and hanger assemblies Although, hangers in a multi-beam system (as were presently in the Pinawa Bridge) have a high degree of redundancy, the same assemblies used in a two-girder framing system offer no redundancy.

3.3 Rocker Bearings

It was observed during the DSSS that only 5 out of 20 rocker bearings were tilting in the correct direction (i.e. contracting when temperatures were below 0oC). Resetting of the rocker bearings is required if the measurement or movement analysis indicates that the rocker bearings have reached or exceeded the allowable tilts at any given temperature (Thompson, B. 2008). Therefore, remediation of the rocker bearings by resetting, blast cleaning, and recoating was required to provide proper function. Blast-cleaning of the pintle holes was also recommended to ensure that after rehabilitation, the bearings are seated properly Furthermore, if the option of replacing the pin-hanger system with a continuity splice was proposed, the fixity of the bridge would be altered, thereby requiring significant modifications or replacement of the bearings at Piers Nos. 1 and 5 as well as replacement of the abutment bearings.

3.4 Expansion Joints

The open joints over both the pin hanger connections (finger joints) and the abutments (sliding steel plate joints) promoted deterioration of the underlying deck slab and steel girders and steel rocker bearings as both joint details allowed salt laden water to leak onto underlying components. This resulted in corrosion of the girder webs and bottom flanges, specifically at the hinge locations and in corrosion of the abutment steel rocker bearings. To alleviate this problem, replacement of the existing expansion joint system at the hinge locations and at the abutments was necessary.

3.5 Girder Coating

The condition of the girder coating varied from light to very severe deterioration over the length of all girders. The most severe deterioration typically occurred at the hinge locations as well as at the bottom of bottom flanges and exterior girder webs. The existing coating was lead based and therefore required negative pressure containment during removal. It was necessary to investigate whether it was more cost effective in the long run to paint the entire girder length or paint the girder sections only at the hinge locations.

3.6 Deck Slab

Tests for water soluble chloride content at or near the level of average reinforcing depth of the bridge deck, curbs, and west abutment indicated that chlorides had surpassed the threshold value and active corrosion of the reinforcement was taking place. Based on the DCCS report, rehabilitation of the bituminous wearing surface and deck slab was required. Therefore, in order to ensure that there were no implications on user costs, rideability, structural adequacy, and public safety, it was necessary to consider durability and longevity of design when determining the deck rehabilitation works.

3.7 Staging and Detours

Two different options were discussed with the project stakeholders for traffic detours during rehabilitation works. Alternating lane closures allowing traffic on one half of the structure and construction work on the other half and full closure of the structure during rehabilitation works were the two options considered. Although the option of lane closures increases bid prices by staged construction being required, the full closure option was not feasible to MIT because emergency services, local stakeholders, and residents would have been significantly impacted as the only available detour is 35 km in length. Therefore lane closures and working on half of the structure at a time was the prudent option.

4 PRELIMINARY DESIGN OF REHABILITATION

Rehabilitation strategies determined with MIT and input from local stakeholders were evaluated in this phase for feasibility and cost effectiveness. The rehabilitation works required to cost effectively extend the service life of the bridge for a period of 25 to 40 years were determined and then developed during the preliminary design.

4.1 Determination of Rehabilitation Works

4.1.1 Superstructure Concrete Works

OSRM recommends that for concrete bridge decks reinforced with uncoated reinforcing steel, concrete removal criteria can be based on half-cell corrosion potential measurements where the average chloride content at the reinforcement level exceeds 0.05% by mass of concrete. Based on the data provided in the DCCS report, the area of concrete to be removed below the reinforcement was estimated to be 68% of the total deck area. This estimate of removal was comprised of areas with high corrosion potential readings and areas of delamination and spalls outside the high corrosion potential areas. Based on the extensive area of concrete that was required to be removed, as well as the scattered locations of these areas on the deck slab, it was recommended that as a minimum, a partial depth deck replacement on the entire deck area be completed along with replacement of the top mat of reinforcement.

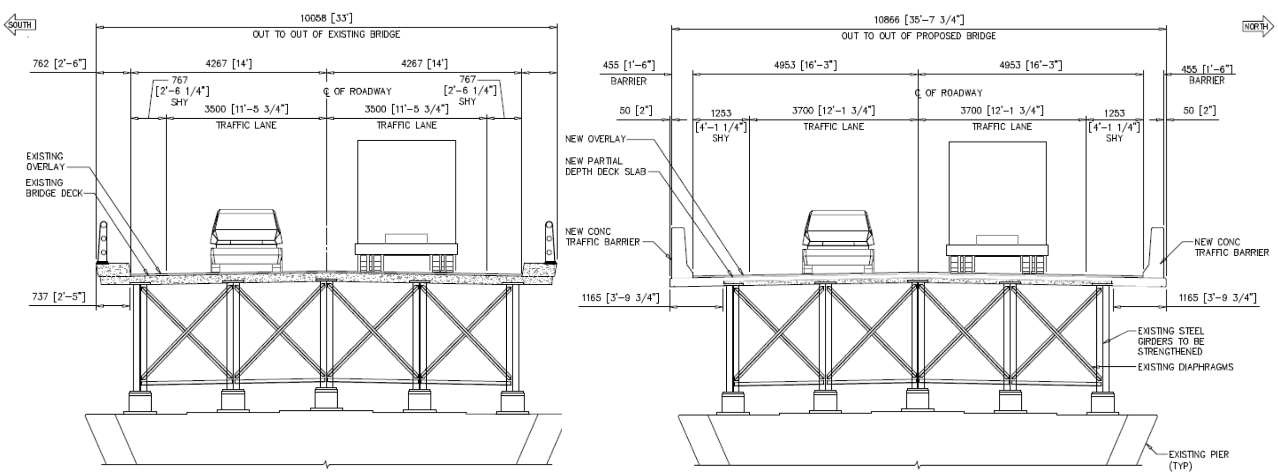

Figure 2: Original Bridge Cross-Section (Left) and Option 1 Bridge Cross-Section (Right)

Three options for bridge deck widening were developed to a conceptual level to satisfy the requirements of construction staging, geometric design criteria, and/or active transportation requirements: the first option featured two 3.7m traffic lanes with 1.25m shoulders; the second option featured two 3.7m traffic lanes with 0.5m shoulders and a 1.2m sidewalk; and the third option featured two 3.7m traffic lanes with 1.1m shoulders and a 1.5m sidewalk. Option 3 required adding an additional girder line and widening the substructure, and therefore this option was not considered feasible. Option 2 was not preferred because it did not satisfy the current GDC requirements. Option 1 was selected as it satisfied the GDC requirements and maximized the clear roadway width without adding an additional girder line or substructure widening. See Figure 2 for details. See Figure 2 for details.

4.1.2 Girder Rehabilitation and Strengthening

Girder strengthening was determined to be required as rating factors of less than 1.0 were calculated for the proposed design vehicles and loads. Bolted cover plates on the bottom flanges and on the webs at insufficient rating factor locations and areas of very severe section loss were recommended.

A fatigue check was completed in accordance with the MBE during the preliminary design. The plate girders, girder welds, splice bolts, and girder base metal were checked for fatigue. Increasing the width of the deck increased the live load distribution factor on the exterior girder. The increased moment caused the stresses in the girder steel go beyond the limits of infinite fatigue life in some girder sections. A finite fatigue check was undertaken and areas with less than 30 years fatigue life remaining were strengthened. Strengthening of the splices and the butt welds on the exterior girders was also required due to fatigue issues. The bottom splice plates and bolts were replaced to increase the fatigue life of the existing bolted girder splice locations. New splice plates were installed across the butt welds to increase the fatigue life of the joints between different flange thicknesses. The installation of catcher beams under the hinges was also recommended as it will provide more redundancy to the pin and hanger assembly (Silano, 1993).

Recoating the entire girder length or partially recoating areas of severe deterioration were both considered. Recoating was proposed to occur in accordance with MIT's approved three coat paint system. For the partial length option, it was proposed that 10m lengths on either side of the hinges be recoated. Environmental containment during sandblasting operations would be common to both options, including precautions required for containment and disposal of lead paint. Even though recoating the entire girder length is initially more expensive, the advantages include a longer service life, overall uniform quality and aesthetics of the superstructure steel, and lower future repair requirements for structural steel.

4.1.3 Approach Roadway

Improvements to the connecting roadways were also developed. The approach roadways and east and west embankments, located on causeways, required widening to satisfy current MIT roadway design widths. The approach roadway pavement needed to be reconstructed to accommodate the widened bridge and approaches. The approach roadway was reconstructed with asphalt pavement in the travel lane and granular shoulders. New approach guardrails and end treatments were constructed. The new guardrail was updated to meet current design lengths. The ditches were reshaped to improve drainage.

4.1.4 Financial Analysis

In addition to estimating the total costs, a life cycle cost analysis (LCCA) was performed for the deck widening steel girder recoating options. The LCCA assessed the overall cost of different solutions based on removal methods of the existing bridge deck, estimated life span of the proposed bridge deck, and the expected maintenance requirements. The variables for this analysis included overlay type, reinforcing bar type, and the length of girder recoating. Black, galvanized, and MMFX-2 micro-composite steel were the types of reinforcing bars considered. Concrete and asphalt were the different types of overlay considered.

The design service lives for both rehabilitation options varied from 25 years to 50 years while Net Present Value (NPV) analysis was carried out over a 30 year period. It was assumed that the rehabilitated bridge will be completely replaced at the 50-year mark and therefore would exceed the study period. As such, a salvage value was assigned to options which outlived the 30 year period. The maintenance cycles for the different options were selected based on the initial mode of rehabilitation. The option of replacing the entire superstructure was also included in the analysis. Based on the analysis, the option with a 150mm thick partial depth deck replacement reinforced with MMFX-2 micro-composite reinforcing bars was the most cost-effective option.

Entire length or partial length girder recoating options using a three coat paint system were also evaluated using the LCCA. Since a platform will have to be built by the Contractor to repair and strengthen the girders along the entire length of the bridge; it proved to be more practical and cost-effective to recoat the entire length of the girders.

4.1.5 Construction Staging

As discussed in Section 3.8, full closure of the structure was not feasible during bridge rehabilitation. Therefore, the rehabilitation works needed to occur in two phases. Temporary signals, signage, and advance advertising of lane closures were utilized to restrict traffic to one lane. Emergency Measures Services were notified in advance of any bridge or roadway closures.

5 DETAILED DESIGN OF REHABILITATION

The detailed design phase of the project included reconfirming the proposed bridge rehabilitation strategy and details and completing detailed structural design for the structure rehabilitation. Construction drawings and tender documents were produced along with construction staging drawings and associated traffic accommodation plans, including sign plans. Detailed construction schedules and identification of probable construction sequencing/ techniques were prepared for all tender packages. Load rating of the rehabilitated structure was completed after the detailed design phase.

5.1 Design Modifications

The detailed design generally followed the recommendations of the Preliminary Design Report. However, the final design incorporated some design elements that that were modified during this phase. The major modification was amending the recommendation of installing catcher beams underneath the pin-hanger system. It was assessed that presence of a catcher beam will induce a dynamic impact factor in the range of 2.6 to 2.8 as the load gets transferred from the existing link assembly to the secondary catcher beam support system (Kulicki, J. 1991). To meet the estimated toughness requirements due to the higher impact factors, higher grade of steel would have been necessary. In addition, by the introduction of the permanent catcher beams, the load transfer mechanism will have been altered and it may have induced additional stresses in the girders thereby necessitating additional strengthening of the girders within the catcher beam region. Historically, the catcher beam system for the pin-hanger connections were typically designed for a continued service life of no more than three years. Within this period the pin-hanger connections were inspected and repaired.

Furthermore, installation of catcher beams under the bridge girders would have altered the bridge profile thus requiring a fresh submission for navigational approval and further delaying the project. Therefore, based on the above reasoning it was decided to replace the pin-hanger system with continuity splices and the existing finger plate expansion joints at the hinges were removed to make these locations continuous. Also, based on the pin hanger systems being replaced with a continuity splice and the joints being removed at the new continuity splice locations, this necessitated new modular deck expansion joints to be incorporated as superstructure expansion will only be accommodated at the abutments after the rehabilitation works were completed. All existing rocker bearings except those at Pier No. 3 were replaced with new uni-directional expansion pot bearings.

5.2 Continuity Splice Design

The challenges associated with designing a continuity splice during partial depth deck slab replacement while permitting only partial lane closures included:

- Supporting both sides of the hinge with an alternate, below deck, system during pin-hanger system removal;
- Provision of varying gap widths during continuity splice installation to account for temperature variation and original installation tolerances; and
- Sequencing of works associated with repairing deteriorated existing structural steel, girder strengthening, and continuity splice installation.

As an alternate, an “above deck” supporting system was proposed by the design team to be included as a conceptual option in the detailed design drawings, however, it was decided that only a below deck temporary support system should be depicted as there was risk associated with traffic colliding with the support system. An “above deck” system may have been simpler to install, but with the requirement to maintain one lane of traffic at all times on the bridge, the “below deck” support system method was more preferable in terms of construction staging sequencing and traffic control. Ultimately, the final design decision on how to support both sides of the hinge with a “below deck” system during pin-hanger system removal was left to the Contractor. The conceptual temporary girder support design provided in the construction drawings is shown in Figure 3.

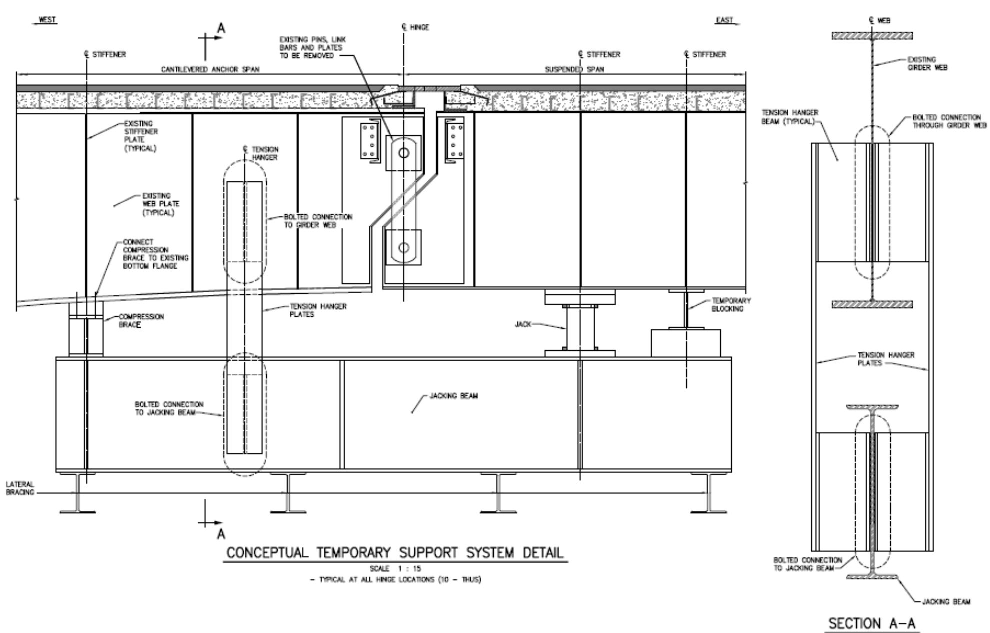

Figure 2 –Conceptual Temporary Support System Detail

To account for the varying gap widths, the continuity splice plates were sized for the maximum predicted hinge gap width, based on previous measurements and the anticipated time of year for installation. These plates could then be field cut, on one side only, in order to ensure they would fit. The bolt holes were all field drilled to allow maximum flexibility in placing the continuity splice plates.

The sequencing of the construction works is discussed in Section 6.

5.3 Bearings Design

The existing fixed rocker bearings at Piers Nos. 1 and 5 were required to be replaced with expansion bearings to accommodate the continuity splice being installed and the bridge being continuous between the abutments. The expansion bearings at Piers Nos. 2 and 4 were also replaced because of two factors: they would now experience greater movement than originally designed for and the cost-benefit comparison was greater for replacement than refurbishment.

Pot bearings were chosen to replace the rocker bearings at the abutments and the piers. The new pot bearings were shallower than the existing rocker bearings. Grout pads and bearing pedestals were constructed at the abutments and piers, respectively, to accommodate the height differential between the pot and rocker bearings.

6 CONSTRUCTION

The general staging of the works included the following:

- Phase 1: Steel works including abutment jacking beam replacement, bearing replacement, continuity splice installation, and steel strengthening works; and
- Phase 2: Concrete works including partial depth deck replacement on half the bridge, including approach road works (split into two stages).

Sequencing of the bridge construction to change the bridge to a fully continuous structure was required. The required order of work for this portion of construction was:

1. Replace jacking beams at both abutments (due to existing jacking beams having insufficient capacity for abutment bearing replacement);
2. Jack and replace all bearings except at Pier No. 3 and install bottom flange lower steel strengthening plates on exterior girders at Pier No. 5; and
3. Install continuity splice including temporary hinge support systems.

Figure 4 - Temporary Support System

The additional steel works included in Phase 1 could be split into two stages, provided they were completed prior to the concrete works being completed above the girder lines being strengthened. Additional staging restrictions included in the contract documents required that all structural steel works was required to be completed under dead load only. This necessitated moving traffic back and forth between the two halves of the bridge. Additionally, completion of continuity splice installation on girder lines A, B, D, and E was required to be completed prior to installation on girder line C (girder lines lettered from north to south).

During construction, the contractor requested permission to begin bridge deck demolition works prior to completion of continuity splice installation and bridge strengthening works. To assist the contractor in achieving the construction schedule, the capacity of the deck and girders at varying stages of completion were calculated and compared to the contractor's proposed construction loads and load positions. Permission to proceed with completing the concrete demolition and steel works concurrently was granted provided specific staging requirements were followed. As the contractor and his subcontractors were completing work concurrently and due to the site spatial constraints, these staging changes required thorough communication and monitoring between MIT and the Contractor as to what works can occur simultaneously to ensure that the reduced capacity of the deck and girders is not compromised.

After removal of the existing bearings, it was noted that the girder bottom flanges were deformed due to the original construction welding heat and the applied loading during the service life of the structure. The sole plates for the new pot bearings also had a larger footprint than the existing bearing sole plates, creating difficulties in achieving 100% contact area between the bottom flange and the sole plate mating surfaces. Remedial repairs to accommodate this issue included using weld filler bars to span excessive gaps (greater than 1.6mm) and completing epoxy injection to fill the void, preventing moisture/debris ingress and creating 100% contact area.

7 CONCLUSION

The Pinawa Bridge project was a major bridge rehabilitation and widening project, which included a staged pin-hanger to continuity splice retrofit that fundamentally changed the behaviour of the bridge, which occurred with traffic continuing to use the bridge during all stages of construction.

Figure 4 – Completed Structure

8 ACKNOWLEDGMENTS

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