Use of Link Slabs in the Highway 17 Over Battle River Bridge Rehabilitation

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ABSTRACT

Project achievements were:

Eliminating all four expansion joints using innovative link slab and semi-integral abutment details. To our knowledge the link slabs in this bridge are the first that used hooked steel fibres in an exposed concrete deck in Saskatchewan/Alberta. The link slabs are performing well after 3 years in-service;

The service life of the 56-year old structure was extended by 40+ years and reduced the carbon footprint of the bridge by salvaging most bridge elements.

Leaking water through bridge deck expansion joints is one of the leading causes for bridge deterioration and results in high priority, costly, and challenging repairs to the bridge elements below them. Using life cycle cost analysis, semi-integral abutments and link slabs at the piers were the most practical and cost-effective solutions.

Although many jurisdictions utilize semi-integral abutments to eliminate abutment joints, the use of link slabs to eliminate pier joints does not share the same popularity. Although the link slab concept is not new, a lack of historical performance data has made some bridge owners reluctant to use this concept. Through our extensive research and careful design, we encouraged the clients to proceed with link slabs.

In additional to the semi-integral abutment conversion and link slab installation, partial depth deck rehabilitation and deck widening, girder strengthening to support new standard design vehicles, upgraded bridge and approach railings to current standards and the superstructure was lifted to facilitate substructure repairs and a bearing replacement.

Some of the unique, challenging, innovative, environmental, and sustainable achievements on this project were:

- Eliminating all expansion joints using innovative link slab details. To our knowledge, these are the first link slabs that have used hooked fibre reinforced concrete in the exposed concrete deck. These details will significantly improve the exposure condition of the underlying elements from chlorides and moisture. This will extend the service life of the bridge and reduce future costs. After being in-service for three-years, the link slabs are performing well with no visible signs of leakage.
- Reducing the carbon footprint of the bridge by salvaging most of a 56-year old structure rather than disposing of it and fabricating new bridge elements. The existing elements were repaired, modified, or strengthened to improve their condition, functionality, and service life.

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1 INTRODUCTION

The bridge spans the Battle River on Saskatchewan Highway No. 17 which is actually located in Alberta.

Figure 1 Location of the Highway 17 over Battle River Bridge. 1

Construction of the original bridge was completed by Alberta Transportation in 1960. The bridge consisted of three spans of precast post tensioned concrete I girders commonly known as PO girders. Spans one and three are 19.05 metres (62'-6") and span two is 23.93 metres (78'-6"). All spans are simply supported at the abutments and pier. Cast-in-place concrete pier walls supported by cast-in-place concrete pile caps and three rows of untreated timber piles support the bridge. Cast-in-place concrete abutments are supported by steel piles. The bridge is capped off with a conventionally reinforced cast-in-place concrete deck and a 50mm lift of asphalt wearing surface. Unsealed sliding steel plate expansion joints were installed at the piers and abutments. The protection system for the deck reinforcing was limited to approximately 25mm of concrete clear cover.

Because this bridge was initially constructed with minimal protection systems, active corrosion was identified during the testing program in 1979, only 19 years after the structure was built. The bridge was re-tested in 1980 and the areas of active corrosion increased; therefore, a first generation rehabilitation was scheduled. In 1982, 22 years after this structure was put into service, a major rehabilitation was completed and consisted of the following repairs:

- The original asphalt and the top 6mm to 10mm of the concrete deck were removed;
- Delaminations and spalls were observed on the deck, which resulted in several repairs, some of which were full-depth;
- A new high density concrete overlay (HDOL) was poured on top of the original concrete deck to add protection to the deck reinforcing, provide a smooth wearing surface, and increase surface drainage;

The new HDOL thickness was 50mm and 94mm at the curb and crown respectively. The new HDOL provided an average concrete clear cover of approximately 100mm to the deck reinforcing; and,

- Sealed expansion joints were installed at the piers and abutments; and reinforced elastomeric expansion bearings were installed at the south pier 1 for both spans of girders and at the north pier for the girders in the northernmost span.;

This structure received half-cell testing in 1983, 1984, and 1988. The test results from 1983 were unreliable, but the results from 1984 and 1988 indicated active corrosion in 22% and 32% of the deck area, respectively. This testing initiated second rehabilitation in 1989, which consisted of the placement of a thin, flexible polymer wearing surface. The new wearing surface was intended to waterproof the deck and provide additional skid resistance. There was some cracking found in the HDOL at this time and some of these were sealed.

After the 1989 rehabilitation, this structure continued to receive regularly scheduled half-cell testing, which is summarized below:

- 1993 – 13% of the deck area had active corrosion;
- 1997 – 17% of the deck area had active corrosion;
- 2001 – 18% of the deck area had active corrosion;
- 2005 – 24% of the deck area had active corrosion;
- 2009 – 39% of the deck area had active corrosion;
- 2012 – 45% of the deck area had active corrosion;

From 1988 to 1993 the area of active corrosion decreased from 32% to 13%, which suggests the rehabilitations completed in 1982 and 1989 had a significant positive impact on the service life of this structure. Since 1993, the deck area with active corrosion has steadily increased, which has prompted the second generation rehabilitation outlined in this report. Alberta Transportation's 2012 half-cell prediction model recommended this rehabilitation to occur between 2014 and 2019.

Over the past few years this structure has been plagued with expansion joint defects, which have resulted in public safety hazards. Deterioration of the concrete deck adjacent to the expansion joints have caused the embedded angles and plates to detach. The loose expansion joint components and corresponding potholes are significant safety concern, which has required several emergency repairs by SMHI and has influenced the timing of this rehabilitation.

2 REPAIR OPTIONS

The repair history noted above and the need to extend the life of this structure required a few rehabilitation options to be developed and then compared using life cycle costing analysis. Four options were analyzed and are presented below.

2.1 Option 1 Do Nothing

In this strategy, no repairs are completed until the critical element reaches the point at which it is no longer able to safely support traffic. Based on the condition survey and service life predictions, it was determined the critical element to be the substructure. Using this strategy, it is assumed that all components of the bridge will be replaced. Although timing for the substructure replacement does not coincide with the replacement of the deck or the girders, it is assumed that the entire structure will be replaced at the time governed by the failure of the substructure. The end of the substructure's service life is expected to occur when spalling around the bearings has progressed to a point in which they no longer safely support the girders. As previously discussed delaminations have been identified around the bearings. The concrete is heavily chloride contaminated, and if the joints continue to leak, the substructure will require emergency repairs and subsequent replacement between 2020 and 2041, with the most likely date of replacement in 2029.

2.2 Option 2 – Reactive Repairs for the Substructure and the Girders with Proactive Repairs for the Deck

For this strategy, no repairs are completed until the critical element reaches the point at which the serviceability of the structure is affected. Based on our condition survey and service life predictions, it was determined the critical element at this structure is the substructure. Using this strategy, it is assumed that all components of the bridge will receive some form of repair. Timing for this rehabilitation is governed by the damage currently in-place at the north pier and abutment and should occur before spalling is evident at the bearings, which is likely not in the too distant future. Estimated timing for this strategy will occur between 2015 and 2021, with the most likely year of 2018. This timing coincides with SMHI's desired rehabilitation time of 2014 or 2015.

Although the deck and girders are not the governing elements for this strategy, the opportune timing for a reactive rehabilitation on these elements should be noted:

- Deck, between 2017 and 2036, with the most likely date being 2026, which is 8 years after the substructure receives a reactive repair. Because there is a significant difference between the opportune reactive repair timing for the deck and substructure, the deck is available to receive a proactive repair.
- Girders, between 2017 and 2024, with the most likely date being 2021, which is 3 years after the substructure receives a reactive repair.

Major work items identified for this strategy include:

- **Safety**
  - The existing curbs and railing system will be removed and replaced; and ,
  - The approach guardrails will be upgraded to meet current standards
- **Function**
  - The deck will be strengthened by removing the existing HDOL and the existing concrete deck to a level below the top mat of reinforcing, installing additional rebar, and increasing the deck thickness to approximately 235mm; and,
  - The girders will be strengthened in shear using external stirrups.
- **Service Life**
  - Deck – The deck repair discussed above will achieve both the desired service life extension and strengthening. This strategy will produce an exposed concrete wearing surface, which will receive transverse sawcuts for skid resistance and silane sealer for added protection. Based on a 235mm thickness, 75mm of cover should be attainable in the new concrete deck;
  - Substructure – Extensive spot repairs will be required on all units, some of which extend below the bearings. This will require the bridge to be lifted and temporarily supported while the repairs are completed. Due to the significant cost associated with lifting this structure it was recommended to excavate all chloride contaminated concrete on the bearing seats and install galvanic cathodic protection to ensure corrosion does not re-initiate and cause damage during the remaining service life of the structure. For all other spot repairs galvanic cells will be installed around the perimeter of the patches;
  - Girders – Isolated spot repairs will be completed to the damaged ends. Galvanic cells will be installed around the perimeter of the patches; and,

- Expansion Joints – The expansion joints will be removed to produce a “jointless” deck. Link slabs will be installed at the piers and the abutments will be converted to semi-integral. To accommodate the thermal movements of the “jointless” deck all bearings will likely be replaced and a joint will be placed at the end of the approach slab.

- **Miscellaneous**
  - Asphalt will be removed and replaced on the approach roadways to accommodate the new deck profiles and elevations; and,
  - Deck and approach road drainage will be upgraded.

For this strategy, once the initial rehabilitation is completed, the substructure and girders may require future spot repairs as chloride contaminated concrete will remain in place. We expect future corrosion and damage will be reduced due to removal of the expansion joints, but based on our previous experiences delaminations may develop, therefore, costs for the potential repairs have been included. The deck is also expected to receive a second intervention within our analysis period, which is expected to occur in 2063. The timing for this second intervention is beyond the desired service life extension, but within our 50 year analysis period.

The timing of this option was found to take place between 2020 and 2041, with the most likely date of replacement in 2029.

2.3 Option 3 – Reactive Repair for Substructure and Girders, Proactive Repair for Deck and Ongoing Preventative Maintenance

This strategy is the same as Option 2, but a high level of preventative maintenance is implemented to the exposed concrete deck to maximize the service life of the concrete cover protection system. This preventative maintenance includes:

- Regular cleaning of the exposed deck and the curbs;
- Regular application of silane sealer to the exposed deck and curbs; and
- Rout and seal cracks in deck as they appear;

Timing for this strategy is the same as Strategy 2, but an allowance for regular preventative maintenance has been included every 7 years. Based on our predictions, the application of preventive maintenance eliminates the need for a second intervention as discussed in Strategy 2. Allowances have also been included to complete ongoing patching of the substructure and girders as previously discussed.

Option 4 – Reactive Repair for Substructure and Girders, Proactive Repair for Deck and Ongoing Preventative Maintenance – Option 1

This strategy is the same as Option 3, but a high level of preventative maintenance is implemented to the exposed concrete deck to maximize the service life of the concrete cover protection system. This preventative maintenance includes:

- Regular cleaning of deck and curbs;
- Regular application of silane sealer to the curbs;
- Rout and seal cracks in the asphalt as the appear; and,
- Replace expansion joint seals

Timing for this strategy is the same as Option 3, but an allowance for regular preventative maintenance has been included every 15 years. Allowances have also been included to complete ongoing patching of the substructure and girders as previously discussed.

Option 2 was the selected repair option for this bridge and the decision to perform this work was agreed to with both the Alberta and Saskatchewan clients.

3 LINK SLABS

A link slab is a reinforced concrete slab that connects simply supported deck spans at the piers of the bridge. The simply supported girders are allowed to rotate while the link transfers forces through the deck. In the traditional approach to link slabs the supports for the simply supported girders act as either rollers or hinges. The existing bridge had supports at the piers which did not allow for the full rotation of the ends of the girder to be used in a link slab. Since all of the joints on the existing bridge were to be replaced and this bridge was to be jointless, changes to the end support conditions of the girders and the bearing was required. The original bearing types and span lengths is described Figure 2.

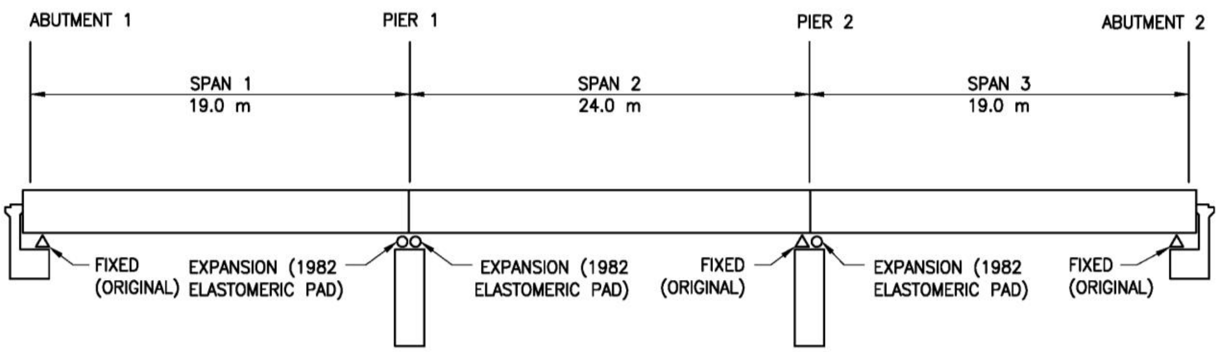

Figure 2. Original Bearing Types.

The length of the concrete girders in the end spans were not equal to the length of the concrete girders in the middle span of the bridge. With the differences the length of the girders at the pier there is a difference in the rotation of the simply supported girders. Also adding some complexity to this bridge is the fact that there are four girders in the 19 meter span and five girders in the 24 meter span of the bridge.

The design of the link slabs over the piers was partially based on the work of Caner and Zia (1). Their work involves analyzing the supports for the bridge as either a hinge or a roller. Since the original bridge had fixed bearings at the abutments, this did not lead itself to the use of link slabs over the piers. Part of the design of the rehabilitation to use link slabs, the end conditions at the abutments needed to change. All girders were to be considered to be simply supported for the design of the link slabs.

Changing the abutments from a traditional abutment wall, abutment seat, wingwalls and slabs to a semi-integral abutment helped the link slab design by changing the bearings at the abutment from fixed to translation bearings. The end condition of girder at the abutment could now be considered to be a roller. The girders that span from the abutment to the piers were considered to have a roller support at the abutment end and a hinge at the pier. The center span girders which are between the piers had their support conditions assumed to be rollers. With this orientation the analysis for the link slabs over the piers used a support condition of RHRR, or Roller, Hinge, Roller, Roller.

The lengths of the girders that met at the piers were not the same, but this was included in the analysis to determine the rotation of the girders at the piers in order to design the link slabs.

Part of the design of a link slab is to ensure that the ensure that the link slab is not made composite with the girders near the joints where the link slab is located. In the areas adjacent to the piers, the deck is to be debonded from the girders. A typical area of the deck where the rehabilitated deck is not to be made composite with the girders is shown in a portion of the design drawings for the bridge shown in Figure 3.

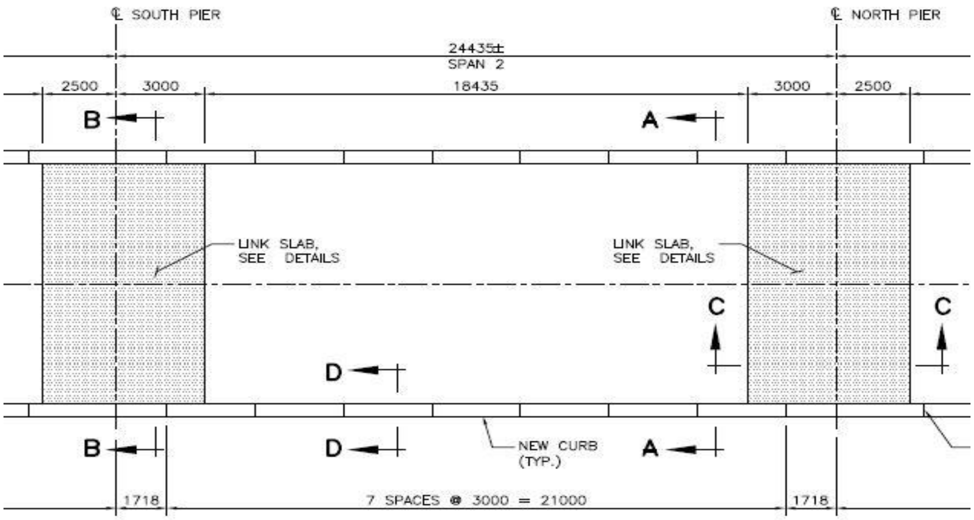

Figure 3 -Debonded areas of the Deck at Link Slab Locations.

A detail on how the deck was separated from the concrete girders is shown in the detail from the design drawings shown in Figure 4.

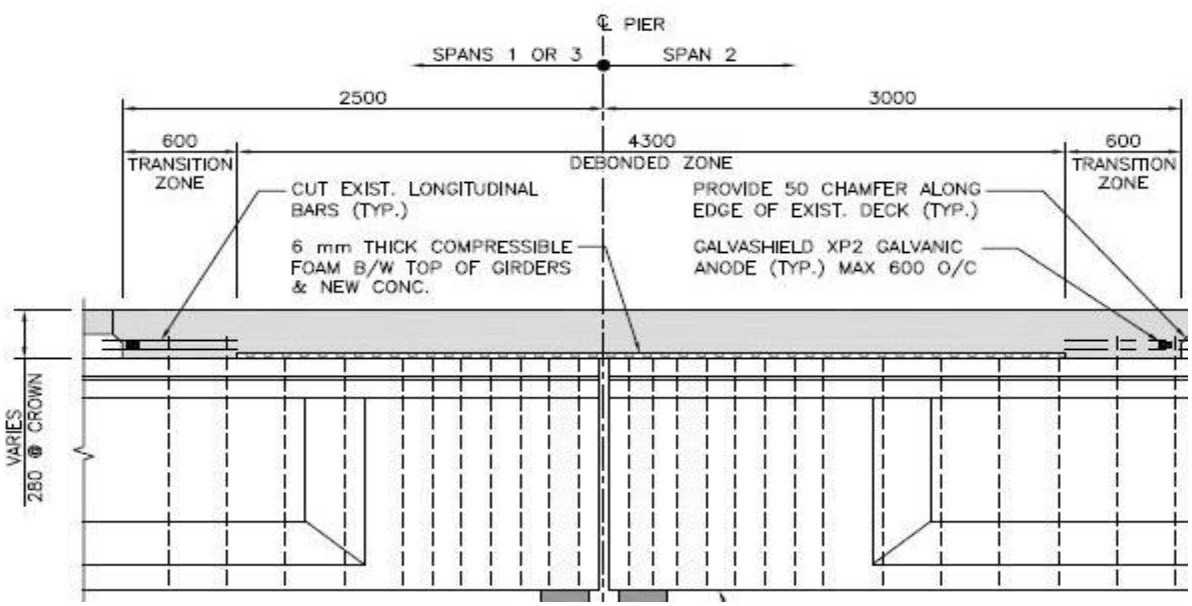

Figure 4 – Debonded Details

Reinforcing details for the new link slabs were determined to try to limit the crack width to a maximum of 0.25 mm. Traditional reinforcing steel could not limit this crack width without large amounts of reinforcing in the deck. Other methods to help reduce the crack width to below the 0.25 mm limit were investigated and include:

- Using different concrete mixes in the deck over the girders and the link slabs;
- Using polypropylene fibres in the concrete mix designs; and,
- Using steel fibres in the concrete mix designs.

The concrete mix designs were evaluated after the tender was awarded and the design was modified to accommodate the concrete supplier, the two owners of the bridge and the steel fibre supplier. A mix which incorporated the Novocon CHE 0960 steel fibres added to the concrete at a rate of 40 kg per cubic metre of concrete would provide the necessary resistance to limit the crack width of 0.25 mm. With this addition of the steel fibres, the reinforcing details for the link slabs from the design drawings is shown in Figure 5.

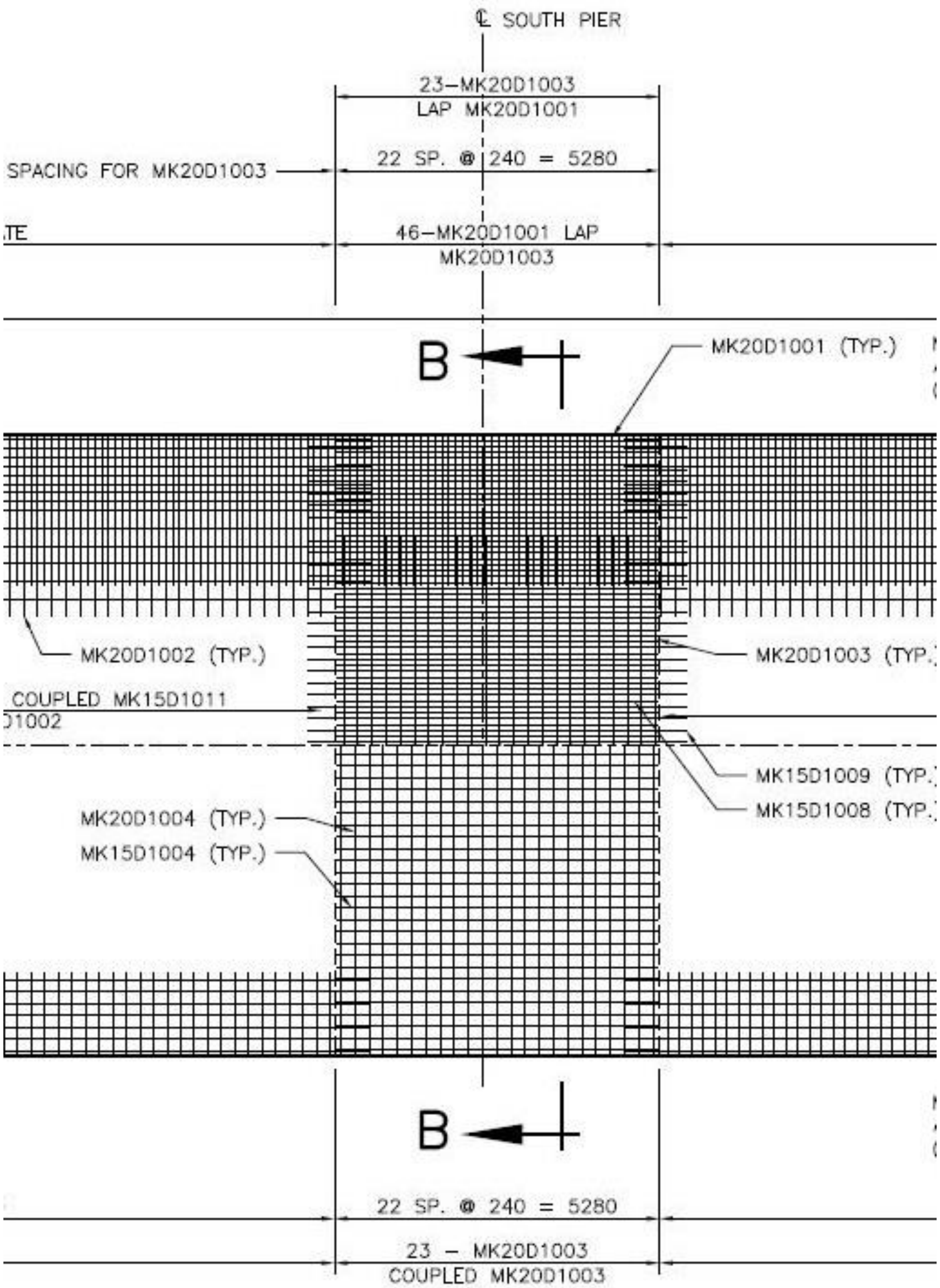

Figure 5 Reinforcing Details for the Link Slabs

The top portion of Figure 5 shows the reinforcing for the top mat of the steel in the deck and the bottom portion of Figure 5 shows the reinforcing for the bottom mat of reinforcing steel in the deck.

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4 CONSTRUCTION

4.1 ***Staged Construction***

With no other bridge crossing the Battle River within an acceptable detour distance, staged construction for the bridge rehabilitation was required. The timeline for the entire bridge rehabilitation was for all the work to be completed in one construction season. Unforeseen issues related to various repairs to the elements of the bridge required an additional construction season to complete. Work on the replacing the bearings and the rehab of the north bound lanes took place in 2016 and the remainder of the rehabilitation took place in 2017.

4.2 Deck Rehabilitation

Part of the rehabilitation involved the removal of the concrete in the deck to below the top mat of reinforcing , the complete removal of the concrete for the link slabs and placement of new concrete was the same for both years of the rehabilitation. A transition between the two lanes of traffic was required in order to produce a deck safe for the traveling public between construction seasons. A detailed description of the rehabilitation of this bridge will not be presented in this paper and it will concentrate on the work for the link slabs. Removal of the concrete at the piers to facilitate the construction of the link slabs can be seen in Figure 6.

Figure 6 Demolition of the Deck at the Piers

The girders in the bottom of the photo are the five girders in the center span of the bridge and girders in the top of the photo are the four girders in the end spans. Reinforcing steel in the deck was salvaged so that new reinforcing could be lapped and joined to the existing reinforcing.

4.3 Link Slab Reinforcing

The areas near the piers required new reinforcing for the deck and the link slabs. The concrete deck was to be debonded from the concrete New reinforcing steel details and the areas of the girders where the concrete was to be debonded from the existing girders can be seen in Figures 7 and 8.

Figure 7 Link Slab Reinforcing

It is hard to see in this photo, but the grey area which appear to be the girders are the foam placed on top of the girders to create the debonded zone for the link slabs.

Comparison between the reinforcing used in the link slabs and the in the new reinforcing steel in the top mat of the new deck can be seen in Figure 7.

Figure 7 - New Top Mat Reinforcing of the Bridge Deck

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*4.4 **Concrete Placement***

With the simply supported portions and the link slabs acting in the tension portion of the continuous deck, separate concrete pours were required. The portions of the deck in the positive moment regions were placed first and allowed to gain a portion of their strength prior to the placement of concrete in the link slabs.

Placing temperature restrictions for the deck concrete mix design limited the placement of the concrete so that the air temperature must not be above 27° Celsius limited the placement to the early morning hours or early evening hours. Traditional concrete finishing equipment was used for both phases of the concrete pours. The concrete was finished with a Gomaco deck finishing machine. With the staged construction, traffic always needed to be accommodated in one lane of the bridge . Special formwork was required to support one end of the Gomaco finishing machine and this formwork was supported by the exterior girder and extended past the exterior girder. The contractor used additional work bridges to perform the final bull float of the surface and to place the deck curing blankets.

Concrete was supplied in traditional concrete mixing trucks from the batch plant to the bridge site. Instead of using concrete trucks to discharge the fresh concrete on the deck or to use concrete pumps to place the concrete, a Telebelt was used. This machine is basically a truck with an extendable conveyor belt. Discharging concrete from the trucks to the belt and then from the belt to the deck has some advantages. Segregation can be reduced. The height of the fall of the concrete can be controlled. Loss of entrained air is minimized as the concrete does not have to travel through a maze of pipes and corners under pressure. The movement of concrete with a Telebelt is not less than a pump. A photo of the Telebelt used at this bridge in shown in Figure 8.

Figure 8 Telebelt.

The concrete is being discharged from the concrete trucks to the belt of the Telebelt at the left side of the photo. The belt then takes the fresh concrete from one belt to the other and then finally discharged through a section of elephant truck at the discharge end shown in the left side of this photo. The deck finishing machine can be seen in the background of this photo.

A finished section of one of the link slabs is shown in Figure 8.

Figure 8 – Recently Placed Link Slab Concrete.

4.5 Concrete Curing

With the use of steel fibres in the concrete mix, no changes were required to the curing of the deck concrete. Seven (7) days of moist curing of the deck concrete was specified in the contract documents. After the concrete had achieved its initial set and the surface would not be damaged by placing blankets on it, the work bridges were used to place the curing blankets on the deck. These blankets had the ability to retain water for up to 14 days but only 7 days was required. The first portion of the deck was placed in October of 2016. Cold weather concreting procedures were needed, and heating was required to keep the concrete warm. Minimal cracking was found on the deck after the curing was completed on the bridge.

The second section of the deck was placed in June of 2017. This year required warm weather concrete procedures. In both seasons, the curing of the concrete blankets were the same.

MONITORING

5 On-Going Monitoring

Stantec and the two owners of the bridge were scheduled to complete an on-going monitoring program on the link slabs for this bridge. The program was to consist of installing instrumentation on the bridge to monitor the rotation of the bridge girders at the piers and in the deck to help determine strains and then loads in the link slabs.

Models for the bridge were to be developed to determine the effect of vehicles on the girders and the link slabs. Calibration of the models using known trucks of known configurations and loads were to be run across the bridge. Ongoing monitoring of the bridge was to take place over several years.

This bridge is to be inspected every two years to the OSIM system and as part of the monitoring, the width of the cracks in the link slabs were to be taken to confirm the results of the model. Budget issues with the owners were encountered and the monitoring of the bridge has not yet begun. It is hoped that funds can be found shortly so that the monitoring program can begin, and results can be published in a future paper.

The bridge has been inspected in 2019 as part of the Saskatchewan Ministry of Highways and Infrastructure major bridge inspection program and no major cracks were found in the bridge deck in either the deck sections or the link slab sections. It is hoped that in addition to the regular OSIM inspection program that takes place on this bridge, a very detailed crack survey will be included. It is hoped that this detailed crack survey will concentrate more on the link slabs. In addition to the crack survey, it is hoped that a model can be created to correlate the loads on the bridge to the crack size in the link slabs.

Acknowledgements

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References

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