MMM Group Limited

LINK SLAB DECK JOINTS

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Paper prepared for presentation at the Session
Bridges – Successes: Let's Build on Them
of the 2011 Annual Conference of the
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ABSTRACT

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There is increasing pressure on bridge designers to minimize joints on bridge decks in order to reduce maintenance and long-term rehabilitation costs. One method of achieving this that is gaining popularity in recent years is the use of concrete link slabs. A concrete link slab is a relatively thin reinforced concrete slab that typically connects simply supported deck spans. It is designed to flex due to girder deflections and also transmit compressive and tensile forces through the deck in conjunction with appropriately designed bearings.

This paper will describe available design methodologies and provide an example of its application for a bridge retrofit. Link slabs are currently being installed in new bridge construction, and also used to replace expansion joints in the rehabilitation of existing structures. The applicable use of link slabs in the field is limited by variables such as girder end rotation from applied loads, bridge skew, and girder depth. Link slabs are designed to flex, however excessive deflection causes potential for the development of wide cracks, exposing the interior steel reinforcement to susceptibility of corrosion. The concrete deck is typically composite with the supporting steel or concrete girders, but is debonded in the link slab region to increase the link slab curvature length, resulting in a reduced slab flexure and minimizing cracking. Although flexural cracking cannot be completely eliminated, water ingress into the cracks can be controlled by the following design considerations: limiting deck crack opening width by limiting end girder rotation; application of waterproofing membrane on top of concrete deck; and use of fibre-reinforced concrete in the link slab.

Introduction

To meet today's challenges of maintaining roads and bridges, innovative methods are being used in the form of new materials and design methods to reduce the amount of maintenance required. Bridges built in the past were typically simple spans, comprised of individual girders with composite decks and joints at the piers and abutments. In the past, these joints have been constructed of compression seals or strip seals with or without protective steel deck panels or steel armoring. Over time, the watertight seals become damaged due to neglect, wear or exposure to the elements allowing water possibly containing deicing salts to leak through the joints onto the girders, diaphragms, bearings and substructure below. Repeated cycles of chloride exposure on concrete causes the steel reinforcement in the girders and substructures to corrode, expanding in volume and causing the concrete to crack and spall. Engineers have long sought methods by which these joints can be better designed, or eliminated altogether. Methods of eliminating deck joints include integral and semi-integral abutments, continuous girders over piers and flexible link slabs.

This paper will discuss the use of link slabs in jointless bridges and its use with Engineered Cementitious Composites (ECC) such as fibre-reinforced concrete and waterproofing membranes.

Background

Link slabs are continuous concrete decks over simply supported steel or concrete girders at the piers. The decks undergo negative flexure as the girders deflect under live load on the spans. The negative flexure results in lateral cracking on the top surface of the deck. Presence of cracks in the top surface of the concrete deck provides a means for water to infiltrate to the reinforcing steel and cause adverse corrosion (see Figure 1).

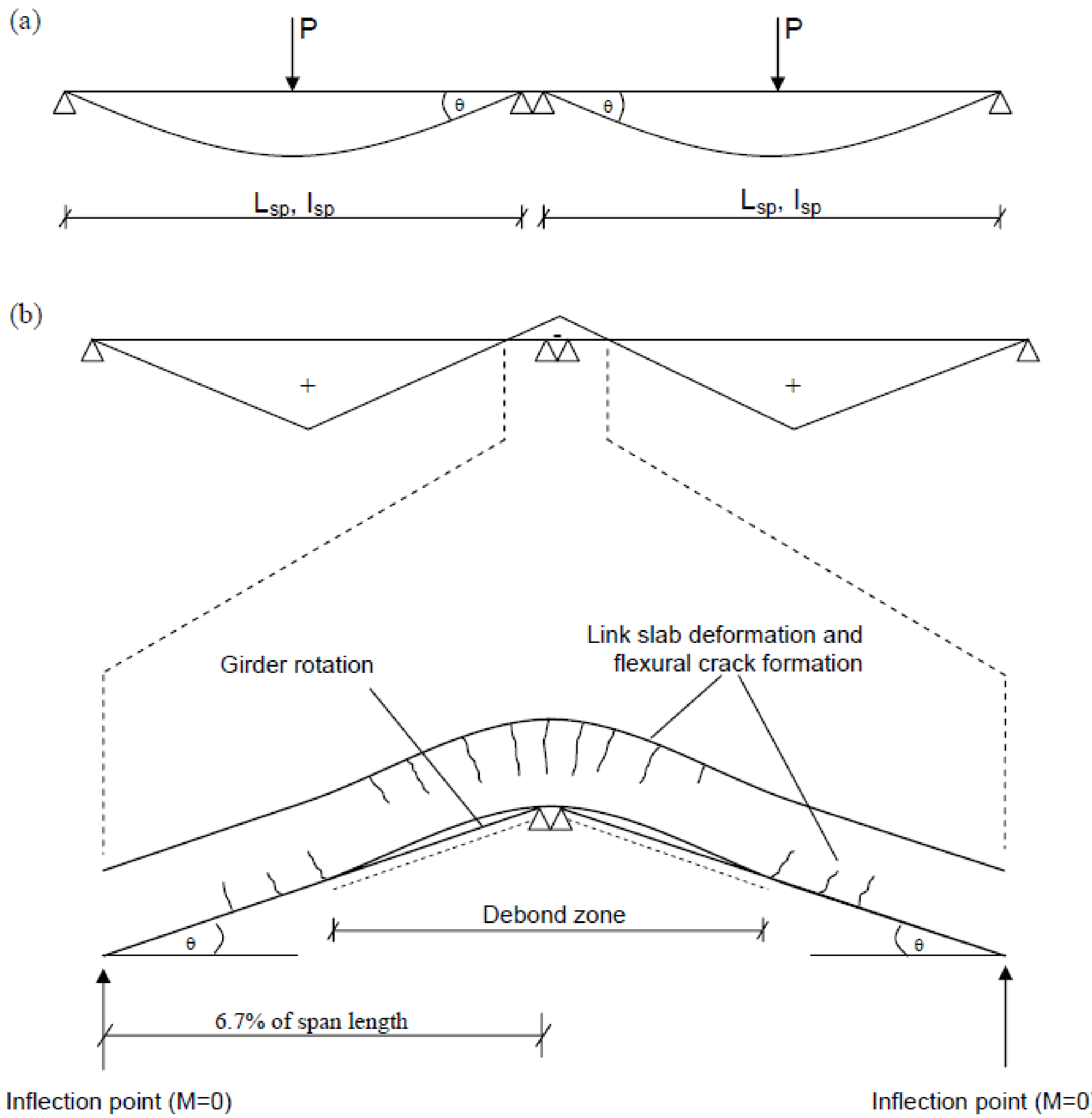

Figure 1) Debonded link slab illustrating flexure of deck in debonded zone

In 1987, the City of Toronto started to use a "flexible link" design to eliminate deck joints in bridges on the Gardiner Expressway. The "flexible link" is a continuous thin concrete slab (150 to 180 mm thick) that spans between haunched beam sections (315 to 375 mm deep) cast on top of the transverse diaphragms at the ends of adjoining girders while the girders are kept discontinuous. [1]

Limitations were placed on the use of the flexible link slab system under certain geometrical and flexural conditions of the structure. These restrictions were introduced primarily to ensure that serviceability limit state design conditions were met when using the standard details proposed in the guidelines. The limitations were related to (a) end rotations of the girders under live load, (b) bridge skew and (c) girder depth. For those cases where the standard flexible link detail cannot be used, a different rehabilitative procedure will be needed and two alternative systems can be used instead:

(1) Convert the simply-supported spans into a semi-continuous deck system for live load by encasing the girder ends in a monolithic transverse concrete diaphragm that is fully connected to the girders by shear studs in order to transfer the negative moments caused by live load and other superimposed dead loads.

(2) Use a similar link slab concept as described above except that the concrete link slab is debonded from the girders for a longer length at each girder end. This provides the link slab with the flexibility required to accommodate the end-rotations of the girders thereby eliminating the need for the haunched beam sections designed to confine any cracking to the link slab. To differentiate from the flexible link slab system discussed above, this new system will be referred to as the debonded link slab system.

Semi-Continuous Deck System
This system is recommended for bridges with a skew of more than 20 degrees or with longitudinal girders that exceed 1.2 m in depth.

Debonded Link Slab System
The second option that can be used to eliminate deck joints in bridges that exceed the limitations of the flexible link slab design is the debonded link slab system (see Figure 2). This system is similar to the flexible link slab concept except that the concrete link slab is debonded from the girders for a longer length at each girder end, thereby providing the flexibility necessary to accommodate the end-rotations of the simply-supported girders. While this eliminates the need for the haunched beam sections and coping of the top flange of the girder (thereby greatly simplifying construction details), it nonetheless requires replacement of a larger area of the deck slab on either side of the pier. The debonded link slab system will be focused on in this paper.

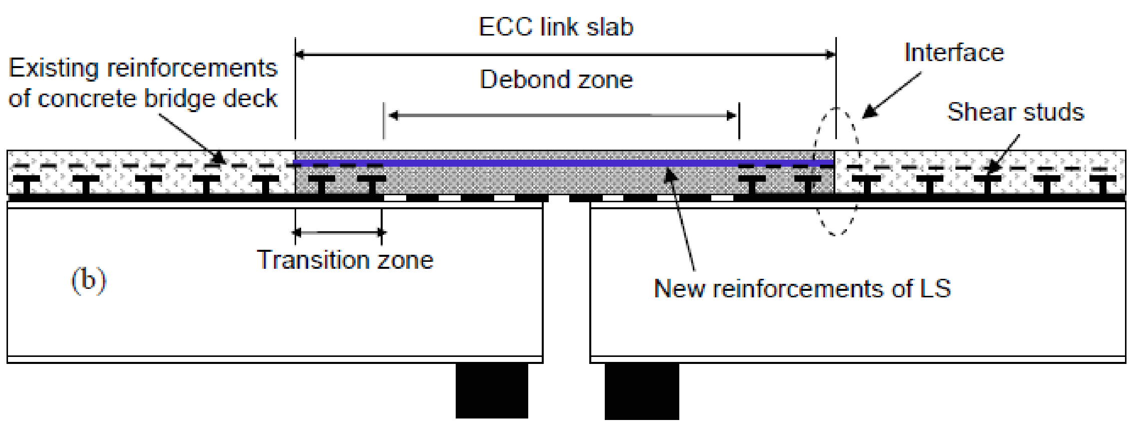

Figure 2) Typical flexible link slab

Methodology

Studies and experiments on link slabs performed by Caner and Zia at the University of North Carolina State University in 1990 developed a simplified method of designing link slabs as described below:

(1) Design each span independently as a simply-supported span ignoring any contribution from the link slab.

(2) Ensure that deck slab is not bonded to the top flanges of girders over a length equivalent to 5% of span length at each end of the adjoining girders. This results in a debonded link slab with much-reduced stiffness.

(3) Determine end rotations of the girders under serviceability limit state (SLS) loads assuming beams are simply-supported and impose end rotations to ends of link slab.

(4) Determine bending moment in link slab due to imposed end rotations, assuming cracked section properties for the link slab.

(5) Design reinforcement for link slab to meet crack control criteria of the Canadian Highway Bridge Design Code (CSA S6-06). [2]

Link slabs are designed to protect the steel reinforcement from corroding by either: using concrete that bends with microcracking, cracks small enough to prevent water from infiltrating to the steel reinforcement; or covering the surface of the deck link slab with waterproofing membrane.

Bendable Concrete

Advances in concrete technology in the past few decades have included adding polypropylene microfibers into concrete mixes. This allows the concrete, normally good at resisting compression but poor at resisting tension, to be able to handle a moderate amount of tensile force and even a certain degree of flexure (see Figure 3). With the polypropylene fibres disbursed evenly throughout the concrete, load is transmitted by bridging of fibres throughout the matrix, and the concrete yields distributed, finer cracking rather than individual larger and deeper cracks (see Figures 4 and 5).

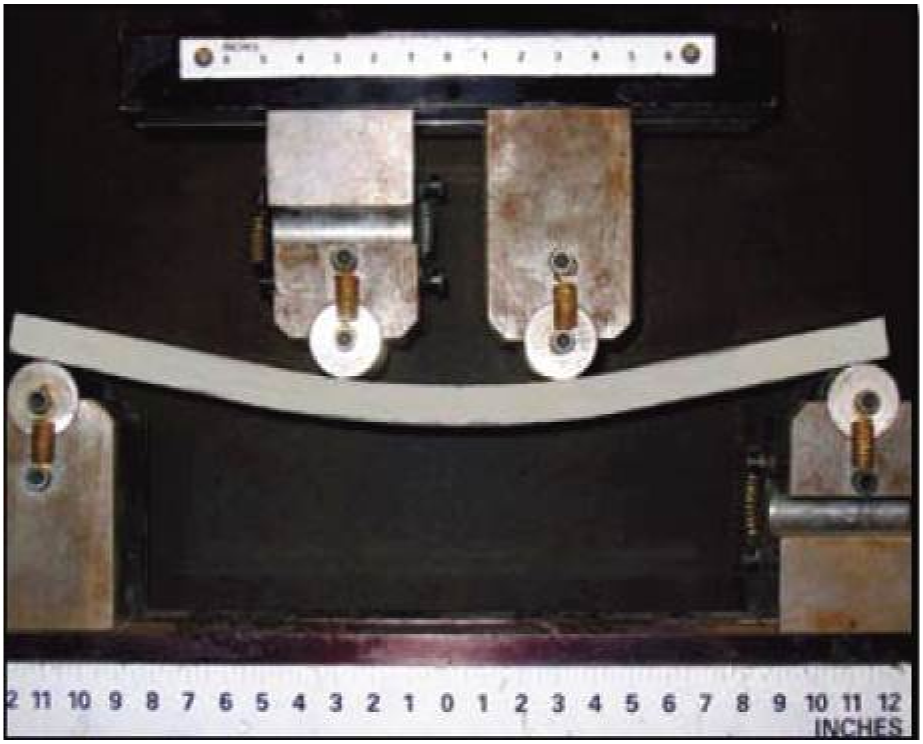

Figure 3) University of Michigan experiment illustrating deformation of unreinforced ECC sample without visible cracking

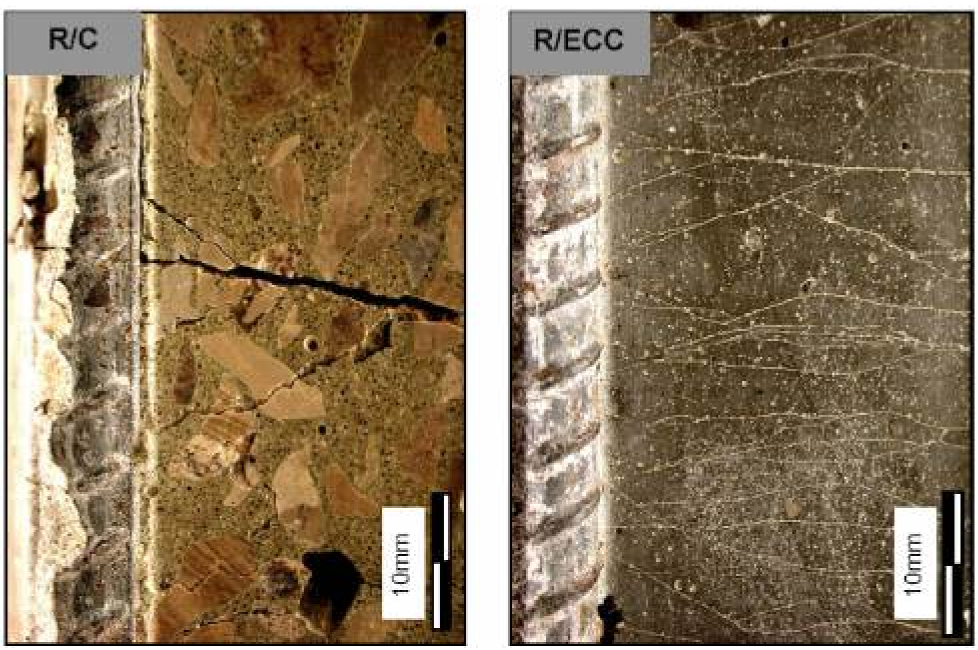

Figure 4) Comparison between cracking due to flexure in normal concrete (shown left) and ECC (right) [4]

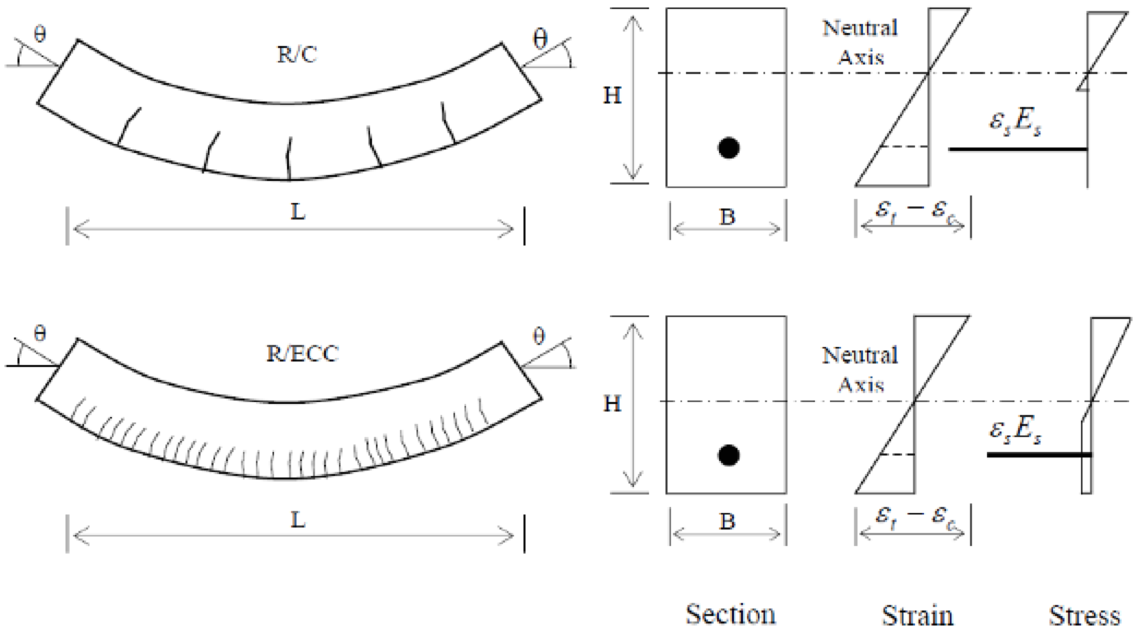

Figure 5) Fewer but larger cracks in normal reinforced concrete beam (top) and more smaller cracks in reinforced ECC beam (bottom)

Certain products are able to achieve 3-5% tensile yield strain compared to 0.01% for normal concrete. Steel reinforcment is still used in link slabs to provide shear and to provide some tensile resistance as fibre reinforced concrete is still relatively expensive to regular steel reinforced concrete.

Link slabs can be used in the remediation of existing bridges to replace old, leaking deck joints. Link slabs can also be used in the construction of new bridges to eliminate any deck joints.

Fatigue Cracking Resistance of Link Slab Specimens

Experiments performed by the University of Michigan on fibre reinforced concrete have been conducted to determine how well concrete is able to withstand the rigors of repeated deflection of the span caused by cyclic loading of the bridge structure.

The following figure illustrates the midspan deflection resulting from cyclic loading of a 28” wide by 128” long link slab between inflection points loaded from 26kN to 103kN. After 10,000 cycles, the specimen did not show any changes in stiffness and no damage was evident (see Figure 6).

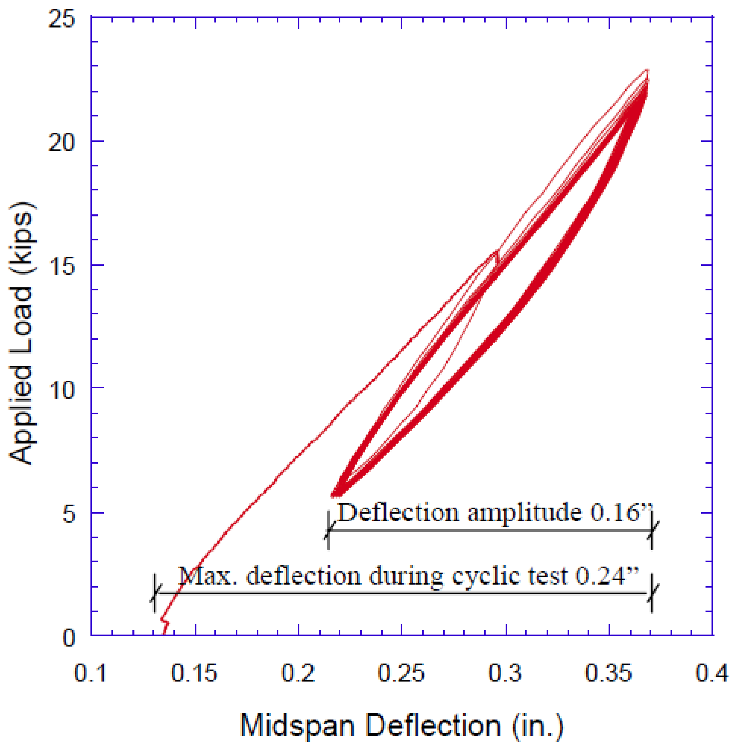

Figure 6) Graph illustrating cyclic loading of ECC link slab [4]

Waterproofing

Another method of protecting ECC link slabs or regular concrete link slabs is to install a waterproofing membrane to protect from water infiltration if the concrete does crack.

Although this alternative is less expensive than constructing the link slab entirely from ECC, there is the cost of regular maintenance or repair of the waterproofing membrane. In areas where snow removal is required, there is the possibility of snow scrapers damaging the membrane as well as environmental damage from rain, snow and UV light, and wear and tear from traffic.

The following is a description of one project in which link-slabs was used to retrofit an existing bridge.

Bow River / CPR Overpass (Westbound) Bridge at km 23.0 TransCanada Highway, Banff, Alberta, Canada

Constructed in 1957, the Bow River / CPR Overpass (Westbound) Bridge is located in Banff National Park (BNP) at km 23.0 on the TransCanada Highway. It consists of seven precast prestressed concrete girder spans of 24.4m (170.8m long overall between centrelines of abutment bearings). The girders support a 14.78m wide (including concrete traffic barriers) 152mm thick reinforced concrete deck with 86mm concrete overlay. The substructure units consist of cast-in-place reinforced concrete abutments and piers founded on timber piles, with the exception of the east abutment which has spread footings.

This rehabilitation project consists of reconstruction of the piers, replacing the deck joints and bearings, and carrying out deck repairs.

The original bridge design included the following:

- 3-3/8 inch (86mm) asphalt overlay
- 6 inch (152mm) concrete deck
- 3 foot 8 inch (1117mm) deep precast prestressed concrete girders
- Steel rocker bearings
- Cast-in-place reinforced concrete piers and abutments
- Expansion joints at piers and abutments comprised of steel angles and rubber seals protected by steel cover plates (see Figure 7).

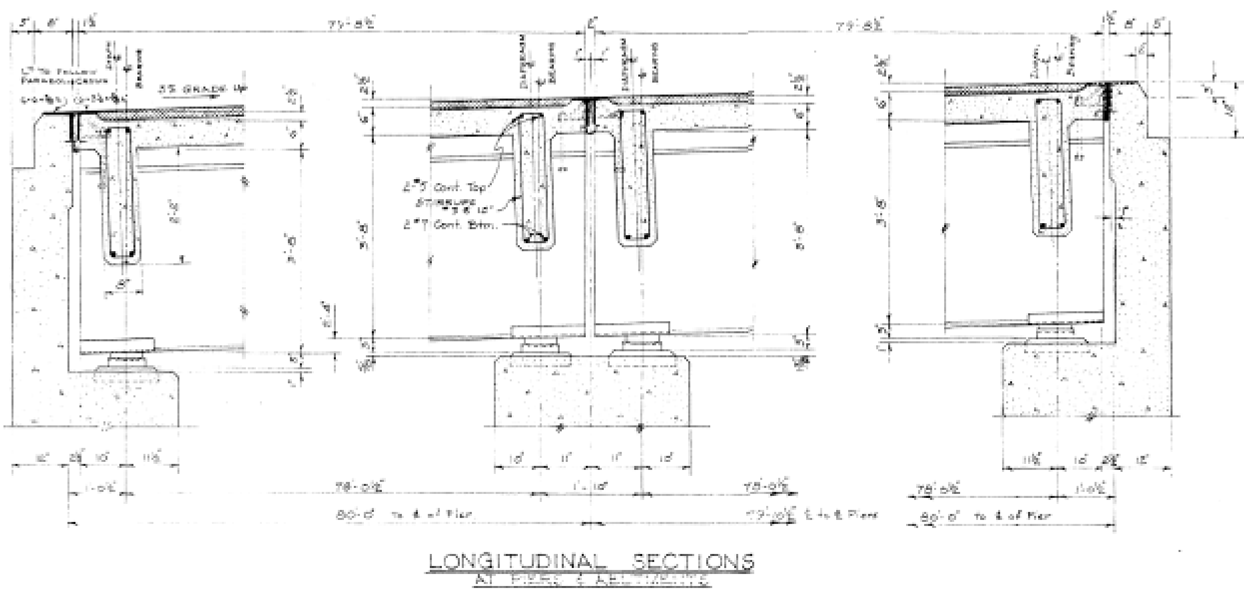

Figure 7) Existing Bow River Bridge deck on Trans Canada Highway km 23.0 prior to remediation

Previous rehabilitation work was performed on the bridge in 1985 which included the following:

- Demolition and removal of existing railing, curbs, and removal of asphalt surface on existing deck.
- Scarifying and removal of top surface of existing deck and installation of a new high density concrete overlay.
- Installation of new expansion joints and new traffic barriers on existing bridge.
- Painting all exposed surface of the new barriers.

- Sandblast and zinc painting of the existing bearings.
- Repair of all existing angle irons at the expansion joints and modify the catch basins at the west end of the bridge.
- Repair of the pier cap and grout pads under all bearing plates.

A condition assessment inspection performed by MMM Group Limited in 2007 determined that the pier caps had reduced load-carrying capacity due to severe corrosion of the reinforcement and heavy spalling of the concrete. Also the girder diaphragms adjacent to the expansion joints were observed to have cracking and spalling. The steel rocker bearings were noted as having fair to serious corrosion and possibly seized in some locations (see Figure 8).

Figure 8) Cracked pier cap with severe corrosion of steel rocker bearings

It was proposed that the expansion joints be removed and replaced with a jointless deck system to prevent future chloride-induced corrosion of the reinforcing steel in the substructure, deck diaphragms and girders.

A debonded link slab option was chosen as the best alternative as it would be relatively quick and inexpensive to construct compared to reinstallation of expansion joints or making the bridge continuous throughout by encasing the ends of the girders with concrete end diaphragms at the piers. (see Figure 9).

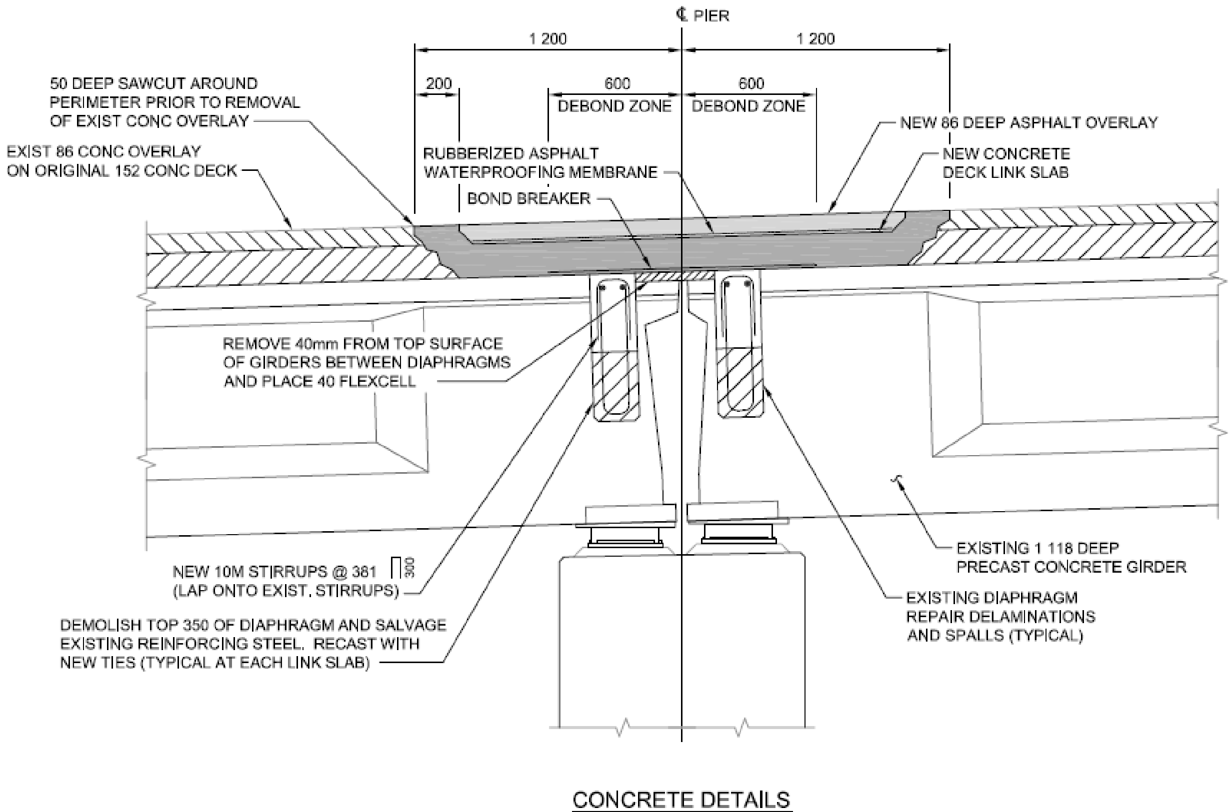

Figure 9) Detail of Link Slab with reconstructed diaphragms, and link slab deck with rubberized waterproofing membrane

The results of the inspection aided in the decision to proceed with link slab replacement of the existing expansion joints. The need for replacement of the expansion joints and the bearings made this structure a viable candidate for the use of link slabs. Computer modeling determined that the girder rotation under live load, the existing bridge skew, and the girder depth met the requirements for a debonded link slab system.

The original design involved pouring the entire link slab thickness of 238mm using fibre-reinforced concrete to allow the deck to flex and mitigate cracking. With fibre-reinforced concrete not readily accessible to the remote location and higher expense compared to regular concrete, the design was revised to 152mm with a water proofing membrane on top and finished with 86mm of asphalt overlay.

The existing structure had spans with one end supported by fixed bearings and the other end with expansion bearings, resulting in all substructure units providing some lateral and longitudinal resistance to deck movement. The rehabilitated structure involved replacing all of the steel rocker bearings with reinforced neoprene bearings at all ends of the girders. With the increased height of the new bearing assembly, the entire bridge superstructure required raising by 27mm. Also changed was fixing all bearings on Piers 2 and 5 into fixed bearings and all other locations into expansion bearings. Calculation checks were performed to ensure that the pier columns at Piers 2 and 5 were capable of carrying all of the lateral and longitudinal loads from the deck. Expansion joints were reconstructed at the abutments and at Pier 3 to allow the structure to be able to accommodate thermal expansion of the superstructure.

An adjacent eastbound bridge structure enabled the vehicular traffic to be diverted during construction, and allowing full closure of the westbound bridge. The pier caps were first demolished and reconstructed. This involved jacking up the entire bridge in sections with hydraulic jacks. The jacks supported the deck while the existing concrete was chipped away, the bearings were removed and the existing shoe plates were sandblasted and metalized (see Figures 10 and 11). Sacrificial zinc anodes were installed between the salvaged pier column reinforcement and the new pier cap reinforcement to prevent new corrosion activity in the reinforcement. A distributed anode system was also applied to the concrete jacketing of the abutment face which exhibited sections of spalling and concrete delamination.

Figure 10) Support system beneath jacked girders during reconstruction of pier cap

Figure 11) Existing sole plates sandblasted and metalized for use with new bearings

After repairs to the piers and abutments were completed, work proceeded to the superstructure. Construction of the link slab required removal of the existing expansion joints and 1.2m of deck on either side of the centerline of pier to allow salvaged reinforcement from the existing deck to be tied into the new link slab (see Figure 12). Sacrificial zinc anodes were used between the existing deck reinforcement and the new link slab reinforcement to prevent future corrosion. The middle 1.2m of the link slab is called the debond zone where a bond breaker consisting of a sheet of polyethylene is placed on top of the girder to separate the deck from the girder. This allows full unrestricted girder rotation, also allowing the link slab to deform in flexure with a gentler curve and reduce the amount of cracking.

The newly constructed concrete barrier had a joint at the midspan of the link slab and was filled with Flexcell to ensure the link slab was free to flex and not be restricted by a continuous barrier. Crash-worthiness of the barriers was achieved by installing galvanized steel dowels in the joint between barrier panels at the link slab which were greased on one end to allow for thermal movement.

A problem encountered was that the concrete diaphragms of the structure were within the debond zone and during the chipping process to remove the deck, the poor condition of the diaphragms caused the diaphragms to crack and spall. The top portion of the diaphragm therefore had to be reconstructed prior to the installation of the poly sheeting and link slab (see Figure 13). Further complications arose when the deck was measured to be less thick than anticipated. The deck thickness was measured to be approximately 200mm thick, and with the link slab designed to be 152mm with 86mm of asphalt topping, this proved to be inadequate. Alternatives were to reduce the asphalt topping to

48mm, or pour the entire 200mm deep link slab with regular concrete and install a waterproofing membrane on top of the concrete wearing surface.

Figure 12) Link slab after chipping away of existing deck

Figure 13) Formwork in place for the concrete diaphragms and link slab deck

An additional change to the original design was the incorporation of a Flexcell layer on top of the ends of the girders beneath the link slab to prevent contact between the corners of the girders and the link slab which may lead to concentrated loading beneath the deck.

The link slab was poured followed by the barriers (see Figure 14), with the finished link slab shown in Figure 15.

Figure 14) Pouring of link slab.

Figure 15) Finished link slab.

Future Considerations

Future design considerations would include a sawcut groove filled with hot poured sealant in the middle of the link slab to induce cracking at that location. Fibre-reinforced concrete would also be a consideration if readily accessible to project site to reduce time and cost of construction of the link slab compared to regular reinforced concrete decking requiring additional time and cost for reinforcing steel installation.

Conclusion

Examples of successful link slab applications have been implemented in Ontario, Canada and Michigan, USA. The benefits of the use of link slabs include reduced costs for maintenance of expansion joints, and less reinforcing steel in the deck resulting in less construction time and cost. Also with the elimination of expansion joints, there is less likelihood of chlorides permeating through the joint and causing corrosion and damage to the reinforced deck and substructure components. The use of link slabs are slowly gaining acceptance as Ministries of Transportation learn more about their benefits of reduced maintenance costs over the lifespan of new or rehabilitated structures.

It is recommended that these link slabs be monitored over their service lives to better determine their long-term effectiveness.

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