Joint Elimination Using Accelerated Bridge Construction Practices on the Indiana Toll Road

Jonathan Lewis, S.E.
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, IL 60062
Phone: (847) 272-7400  Fax: (847) 291-9599
Email: jlewis@wje.com

Jonathan C. McGormley, P.E., S.E.*
Wiss, Janney, Elstner Associates, Inc.
330 Pfingsten Road
Northbrook, IL 60062
Phone: (847) 272-7400  Fax: (847) 291-9599
Email: jmcmgormley@wje.com

Robert D. Ladson, P.E.
Infrastructure Manager
ITR Concession Company LLC
52551 Ash Road
Granger, IN 46530
Phone: (574) 651-2410
rladson@indianatollroad.org

*Corresponding Author

Abstract 240 < 250
Word Count 4961
Figure/Table 9@250 each = 2250
Total 7211 (with 9 figures)
Submission Date August 1, 2013
ABSTRACT

Originally constructed in the mid-1950s, the Indiana Toll Road (ITR) extends nearly 160 miles across northern Indiana and consists of hundreds of bridge structures of varying complexity. A commonly used ITR mainline structure type consists of two parallel bridges, each with three simple spans and carrying two traffic lanes. As with many bridge structures of this vintage, they now suffer from advanced corrosion-related deterioration, particularly in the vicinity of the bridge deck joints. Two of these typical structures were recently targeted for repair by the concessionaire operating the ITR, with the goal of salvaging and preserving the bridge decks in a manner that would lead to an additional 15 to 20 years of service life. A comprehensive evaluation of the bridges was done, including detailed durability analyses and service life modeling to predict the remaining service life of the bridge decks. The durability modeling and condition survey indicated that the bridge decks and existing concrete overlays could be addressed with simple patching repairs and still be expected to perform serviceably in future years. A novel approach for eliminating the bridge deck joints over piers and converting the abutments to semi-integral construction was implemented to eliminate salt-laden runoff and mitigate deterioration to the superstructure and substructure. Repair methods were developed in a manner that eliminated long-term lane closures on the ITR mainline during construction, significantly reducing anticipated traffic delays and improving work-zone safety for the motoring public during peak travel periods.
INTRODUCTION AND BACKGROUND

The Indiana Toll Road (ITR) carries Interstates 80 and 90 across northern Indiana, connecting with the Chicago Skyway to the west and the Ohio Turnpike to the east. Within the project area, the 157-mile system is used by more than 25,000 vehicles daily (1), of which approximately 36 percent are trucks, and provides access to Interstates 65, 69, and 94. Constructed in the mid-1950s, the ITR system includes 333 bridge structures over local roads, highways, railroads, and waterways. Many of these structures consist of multiple simply supported steel girder spans that support cast-in-place concrete decks. The steel beams are commonly supported on steel shoe-type bearings that rest on cast-in-place concrete bents or vaulted abutments. These substructure elements are typically founded on driven steel piles. A common three-span, simply supported structure is shown in Figure 1.

![Typical ITR three-span structure](image)

The superstructure and substructure elements for the vast majority of the ITR structures date to the original construction, with localized patching repairs and latex-modified concrete overlays implemented on many of the bridges in the 1980s to replace the original 2-inch asphalt wearing surface. Some of the original structures have since been replaced, widened, or otherwise modified.

From the opening of the ITR in the 1950s until 2006, the ITR was operated and maintained by either the Indiana Toll Road Commission or the Indiana Department of Transportation (INDOT). In early 2006, the State of Indiana issued a 75-year lease to a private firm, the IRT Concession Company (ITRCC), for the long-term operation of the ITR. In exchange for a one-time lump sum payment of $3.85 billion to the state of Indiana, the ITRCC was given sole operating authority over the ITR until the year 2081. The terms of the agreement enabled the ITRCC to collect all revenue from tolls, but also assigned the ITRCC the full responsibility of operating and maintaining the ITR over the lease period. The ITRCC formally took control of the ITR on June 30, 2006, and since then has invested in bridge replacement, repair and preservation efforts. As is typical for bridges in northern climates, most of the bridge repairs and preservation efforts are focused on addressing or preventing corrosion related deterioration in superstructure and substructure elements.

The high volume of traffic using the ITR and the heavy concentration of trucks make maintenance of traffic a key component for any work on the ITR mainline. Since the ITR carries two of the most heavily travelled east-west interstates in the entire country and large stretches consist of only two traffic lanes in each direction, the ITRCC has established lane closure guidelines that significantly reduce...
anticipated traffic delays and improve work-zone safety for the motoring public during peak travel periods. Conventional construction methods require the use of crossover ramps or temporary structures to maintain at least two open lanes of travel in each direction. As such, accelerated construction methods for mainline repair projects are particularly attractive for ITR structures.

In mid-2012, the ITRCC targeted eleven bridge structures for repair and preservation, and solicited consulting engineering services to assist with the development of a course of action for each structure. Two three-span, simply supported bridge structures over local roads in rural central Indiana were included in the 2012 capital program. This paper explores the repair and preservation approaches used for these structures and also discusses the novel accelerated bridge construction techniques used to minimize disruptions and enhance construction zone safety for motorists on the ITR.

OVERVIEW OF BRIDGE STRUCTURES

Bridge Descriptions

The two structures discussed in this paper were both built in the mid-1950s as part of the original construction of the ITR. Structure 24A carries the ITR mainline over County Road 900E at the border between Porter and St. Joseph Counties, and Structure 25B carries the ITR mainline over Tamarack Road. With the exception of a latex modified concrete overlay installed in 1987, both structures are substantially unmodified from their original construction. Each structure consists of parallel and independent eastbound and westbound bridges that carry two lanes of traffic in either direction. The combined thickness of the concrete overlay and original deck is 12 inches, with the overlay comprising the top 2 inches. The original concrete deck measured 7 inches thick according to the original plans.

The bridge decks are supported on five simply supported wide-flange steel beams. Each bridge consists of three spans, and only the middle span is made composite with the deck slab via channel shear connectors welded to the top flange of the steel beams. There is no mechanical connection between the slab and beams in the end spans. For both structures, the two end spans measure approximately 25 feet in length, and the middle span measures approximately 35 feet in length for an overall bridge length from abutment to abutment of 87 to 89 feet. In the original configuration, each individual beam was provided with fixed shoe bearing at one end and an expansion bearing at the other end. The bearings are anchored to the top of reinforced concrete piers or abutments, which are in turn founded on driven piles. Vaulted abutment slabs extend approximately 13 feet beyond the abutment joints and meet reinforced concrete approach slabs at each end of the bridges. The bridges measure 32 feet 4 inches wide from the inside faces of the parapets and carry two 12-foot-wide travel lanes, with approximately 4-foot-wide shoulders.

Typical Distress

Decades of exposure to de-icing salts and chloride-laden runoff had led to significant localized deterioration in the bridge decks at both structures, especially near the deck joints and on the deck undersides at the locations of transverse through-cracking (Figure 2). Only sporadic cracking and delamination were noted in the concrete overlays and concrete bridge rails for both structures. At many locations, failed deck joint seals had allowed deck runoff to deteriorate adjacent portions of the steel framing and concrete substructure elements (Figure 2). Corrosion buildup and wear at several bearings caused them to “bounce” noticeably under truck loading. Portions of deck slab above the non-composite girders in the end spans were also observed to move or pound against the girders under truck loads.
Investigation and Durability Modeling

Overview

Previously completed investigations by others had targeted the two bridge structures in this study for full replacement of the existing bridge decks, extensive crossover ramps for maintenance of traffic, and numerous other costly modifications. A preliminary review of the previous investigation data suggested that the bridge decks could likely be salvaged, and, with localized concrete patching and joint repairs, achieve an additional service life of 15 to 20 years consistent with the ITRCC goals for these structures. To justify keeping and repairing the existing bridge decks and overlay, a supplemental investigation was performed to better document the condition of the bridges and to obtain data for use in durability modeling of the overlay and deck concrete. This supplemental investigation included:

- Visual and hammer sounding surveys of accessible portions of the top and bottom deck surfaces and concrete substructure elements; and visual observations of the steel superstructure
- Rebar and cover surveys at selected locations of the deck and substructure elements
- Measurements of carbonation depth and half-cell potential (HCP) surveys
- Extraction of core samples from the deck and substructure elements for petrographic examination, strength testing, and determination of chloride penetration depth

The information obtained from the supplemental field investigation was then used to perform durability modeling of the deck and overlay to determine their remaining useful lives. Figure 3 is a compilation of relevant investigation data for a portion of a typical bridge deck.

Since one of the primary goals of a concrete overlay is to protect the deck reinforcing from corrosion, the rate at which chloride contamination is progressing through the overlay and into the deck concrete is the key variable in determining when an overlay or bridge deck should be replaced. If durability modeling indicates that the depth of chloride contamination will not extend to the level of the top layer of reinforcing steel in the bridge deck in a specified time period, it is reasonable to assume that the reinforcing will not corrode and widespread deck deterioration will not develop. Alternatively, if
elevated chlorides are found at the level of the steel (from samples taken through the deck) or appear to be imminent, then the overlay and possibly the underlying deck slab may be near the end of their serviceable lives.

Findings

The field investigation revealed that deterioration in the bridge decks was fairly localized and had not yet affected widespread areas of these structures. Typical distress on the top surface of the bridge deck overlays was limited to isolated cracking on the order of 0.02 inches or less in width, some of which had been previously sealed. Some sporadic areas of spalling or delamination in the overlays were present near the bridge deck joints, but these areas accounted for only 1 to 2 percent of the total deck area. The overlay remained well-bonded to the original deck substrate. Overlay thicknesses in the areas tested were found to vary from 4.25 to 5.21 inches, thus providing between 5 and 6 inches of clear cover to the top layer of deck reinforcing steel, which helped explain the lack of any extensive topside deterioration. Transverse cracking was fairly common on the underside of the bridge deck, and some cracks exhibited staining, efflorescence, or other signs of active leakage. Spalled and delaminated concrete were somewhat common at leaking cracks, but, in total, only about 10 to 12 percent of the slab undersides were found to be delaminated or spalled.

Assessment of the chloride data indicated that elevated chloride levels were confined to the overlay and had not widely penetrated the deck. Localized areas of elevated chlorides were identified at transverse cracks, indicating that active leaks through the overlay and deck were leading to corrosion-related distress on the slab undersides at isolated locations. Further, durability modeling predicted that the concentration of chlorides away from cracks would take many years to accumulate to any significant level. Therefore, provided that the potential for corrosion near overlay/deck cracks was properly addressed and routine maintenance performed, replacement of the overlay for these structures could likely be deferred for more than 20 years. This finding was quite significant in that it eliminated the need for costly deck replacements (and associated traffic control complexities) and enabled the ITRCC to proceed with localized patching repairs to address current distress. In order to address ongoing chloride ingress through existing deck and overlay cracks, it was recommended that the entire deck surface be sealed with silane and high-molecular-weight-methacrylate (HMWM).

Deterioration to the steel superstructure for both bridges was very minor and limited to some localized structurally insignificant corrosion-related section loss near the bridge deck joints and isolated impact-related distortion from vehicles on the local roads underneath the structures. In the end spans, the
deck was observed to move relative to the top flange of the non-composite beams resulting in concrete spalling and fretting rust. The overall coating systems for the superstructures were in poor condition and had reached the end of their serviceable lives.

Substructure deterioration was also fairly isolated and consisted of cracking, delamination and spalling due to runoff through the failed bridge deck joints above. Survey results indicated that each bridge would require about 200 square feet of partial-depth concrete repairs to substructure elements.

**PROPOSED COURSE OF ACTION**

The ITRCC requested that two lanes of traffic be maintained during peak travel periods throughout any bridge rehabilitation projects. Because the deck overlay was still serviceable, extended lane closures could be avoided since most of the proposed repair actions could be completed using short-term closures with proper selection of repair materials and procedures. Therefore, the course of action proposed limited single-lane closures from only March to Memorial Day Weekend occurring Monday through Thursday and that rapid-setting materials and accelerated bridge construction practices be employed. The advantages to this approach included reduced traffic control costs and full lane capacity on high travel days and weekends.

From the supplemental field investigation findings, the following course of action for the primary structural components of these structures was proposed. Because the structures were still in serviceable condition and would continue to be so in the near term; and no improvements in load carrying capacity were planned, the proposed course of action was better characterized as bridge preservation instead of rehabilitation.

**Decks**

Based on the deck investigations, removal and replacement of neither the decks nor the overlays was warranted. For both structures, conventional concrete repairs of the isolated delaminated areas on the deck topside were appropriate. To make the topside repairs more durable than the surficial patches currently in place while supporting traffic after the limited lane closures, a rapid-setting cementitious repair material was recommended, and the repairs were to be anchored in place with new epoxy-grouted dowels or extended below the top bars.

Deck underside deterioration was more widespread and appeared to be caused on all decks largely by 1) chloride-induced corrosion where full-thickness transverse deck cracks have permitted chloride and moisture to penetrate to the bottom bars, or 2) spalling related to movement and corrosion occurring at beam flange tips. Some limited freeze-thaw damage was present at the edges of some of the deck slabs, particularly near the joints. Deterioration proximal to the beams was shallow in nature and could be mitigated by addressing the vertical movement between the slabs and beams. Therefore, the primary repairs to the deck undersides focused on addressing deterioration influenced by through-thickness cracking. Local concrete repairs were specified at areas of the deck undersides showing corrosion-related damage. These repairs would remove the chloride-contaminated concrete from around the exposed corroding reinforcing steel and replace it with a high-quality concrete repair material. Such repairs would likely best be executed by shotcrete methods, provided a qualified contractor and a rigorous quality control plan were employed. Isolated repairs needed for the exterior parapet wall could be completed when the soffit repairs were performed using conventional partial-depth concrete repair methods (e.g. formed concrete repair).

While conventional concrete repairs were appropriate for deterioration near the soffit surface, there was some risk that high chloride levels from the transverse cracks surrounding the top mat bars may cause corrosion to initiate there. However, the rate at which such corrosion was likely to occur was low because of the large cover and because treatment of the cracks on the deck surface would limit future chloride, moisture, and oxygen ingress. Nevertheless, to reduce the potential at the slab underside repair areas for adjacent concrete reinforcement to begin corroding, embedded galvanic anodes were specified.
Discrete galvanic anodes have shown to be effective at reducing ring anode effects and, when installed in the soffit patches, would protect bars in both the top and bottom mats.

After the installation of the deck patches, the topside deck cracks were recommended to be sealed with a silane sealer and HMWM to limit future moisture and chloride ingress. The top surface of the deck would be subjected to a diamond grinding process to remove the immediate surface layer of concrete. This operation would also facilitate removal of the existing asphaltic sealant present in cracks and clean the concrete surface so that the subsequent surface treatments would effectively bond to the concrete. Application of the silane and HMWM materials would then follow directly. Silane sealers do not fill or bond the cracks but they make the sides of narrow cracks (as narrow as 0.002 inches) water repellent. After applying the silane sealer, a HMWM resin would then be swept on the deck. If properly applied, the resin would fill or partially-fill most cracks, limiting deicing solution penetration in the deck. The HMWM has a high solvent capacity that enables it to bond through lightly contaminated surfaces, and the preceding silane application should improve the adhesion of the HMWM to the concrete. Properly filled cracks would remain sealed even when the surface abrades. Sand would be broadcast into the resin before it hardens to improve skid resistance until traffic wear removes any residue.

Delaminations and spalling present on the bridge rail were specified to be repaired with conventional concrete repair methods.

Deck Joints

Expansion joints are notoriously poorly performing bridge elements, and achieving a 20-year service life with common bridge joint seals can be difficult. Therefore, joint repairs beyond routine seal or gland replacements would most likely be necessary within that period in order to adequately control the exposure of the underlying superstructure and substructure elements and to ensure each of these elements can meet their predicted service lives.

While leakage was fairly limited at some of the deck joints, the current work provided an opportunity to retrofit the deck joints. Because of their age and condition, repair of the existing expansion joints did not appear to be a reliable option to extend the life of the deck system. Therefore, two potential approaches for retrofitting the joints were replacement or elimination.

The ITRCC had expressed a desire to eliminate the bridge deck expansion joints and associated maintenance issues from the bridge decks by converting the abutments to semi-integral structures and connecting the decks of adjacent simply supported spans with a link slab. A conceptual approach was developed to remove the end bridge joint at the vaulted abutment and tie the deck together over the pier joints.

Superstructure and Bearings

In general, the superstructure steel was in fair condition with only localized areas of minor section loss. The investigation did not reveal any locations where the deterioration had progressed to the point where it was deemed structurally significant. The age and condition of the superstructure coating suggested that a new protective coating system be included with any superstructure work.

For both structures, all bearings would be replaced to accommodate the joint elimination approach. This would also address the observed vertical movement which appears related to corrosion and bearing wear.

The vertical movement observed between the concrete deck and the top of the steel beams was likely associated with bearing and expansion joint corrosion and wear. Vertical movement between slabs and non-composite steel beams is not uncommon in older bridge structures. This movement did not pose a significant structural concern, but did raise long-term durability concerns for concrete in the vicinity of the affected beams. Injecting a urethane grout into the gaps between the concrete deck and the top of the steel beams was specified to restore intimate contact between the slab and beam, cushioning or eliminating the observed deck pounding.
Substructure

The proposed course of action included conventional concrete repairs to the columns, cap beams and abutments to address the corrosion-related deterioration. Formed concrete repairs were specified to remove the chloride-contaminated concrete around corroded reinforcing steel and replace it with high-quality concrete repair material. Embedded galvanic anodes were also included to reduce the potential for adjacent concrete reinforcement in existing chloride-laden concrete to begin an accelerated corrosion process called ring-anode or halo effect.

The HCP measurements indicated that some corrosion was most likely occurring below expansion joints, but that the corrosion had not advanced sufficiently to cause delaminations as confirmed from our field investigation. Therefore, by either replacing or removing the expansion joints in combination with the application of a silane surface sealer to reduce the exposure of the concrete to chlorides, the rate of corrosion would decrease, reducing the potential for spalls to occur.

DESIGN CONSIDERATIONS

Elimination of existing bridge deck joints in bridges began the 1960s and has been implemented intermittently and with no uniformity since. Limited literature on the subject exists—particularly with respect to converting existing steel bridges to continuous or semi-continuous. A 2002 report prepared by Rhode Island University (2) included a survey of state and province transportation agencies practices on this subject and an analytical, parametric study of commonly implemented conversion practices. INDOT’s Bridge Rehabilitation Techniques Joint Elimination Reference (3)—the agency with design authority for the ITR—identifies considerations before implementing a conversion. Most significant and consistent with the Rhode Island study was the expectation for deck cracking in the link slab used to make the deck continuous. Also to be considered was the need to replace existing steel shoe bearings with new elastomeric bearings and an evaluation of the existing substructure to resist the change in bearing fixity.

In order to move the end joints off the bridge, the abutments had to be converted to semi-integral construction. A review of available literature and standard details from Indiana and other state departments of transportation on conversion to semi-integral abutments was conducted to determine best practices. Findings from this review determined that skews greater 30 degrees and bridge expansion lengths longer than 250 feet can be especially problematic and require further investigation before implementation. Since the structures included in this study were neither significantly skewed nor long, they appeared to be strong candidates for joint elimination and abutment conversion.

Substructure Analysis

To justify the elimination of bridge deck joints over the piers and to convert the abutments to semi-integral construction, an analysis of the substructure was carried out. Although the span lengths for the three-span bridges included in this study were rather short (approximately 25 to 35 feet) and corresponding thermal movements were small, consistent with the joint elimination approach recommendations, full replacement of the original bearings was pursued. Further, with the deck continuous over the joints, but the beams still discontinuous, rotational demands under live load would have to be accommodated by the bearings and piers. This approach was somewhat unorthodox according to the literature in that it is desirable to make the beams continuous with the deck. However, making the beams continuous would have required extensive steel repairs and would have increased the costs of the joint elimination substantially. Therefore, the suitability of leaving the beams discontinuous was confirmed by analysis. In the proposed design, the two piers for each three-span bridge would each support a fixed and expansion bearing. Figure 4 is an elevation view of a typical three-span bridge showing the original and proposed bearing configurations. Rotational demands generated by live load deflections in the span supported by the fixed bearings would have to be accommodated by longitudinal deflection of the piers since the fixed bearings cannot move relative to the piers. At the expansion
bears, the rotational demands would be absorbed by slight longitudinal deflection of the elastomeric bearings.

The piers and their foundations were analyzed for the combined demands from external loads, live load rotation compatibility, and additional longitudinal forces generated by thermal movements. All piers were found to have sufficient capacity for these additional loads. Bearings were sized for the new movement demands and additional live loads from the semi-integral abutments.

![FIGURE 4. Elevation of three-span bridge showing existing and proposed bearing situation (E)=Exist., (N)= New, E=Expan., and F=Fixed](image)

**Link Slabs**

A link slab connecting adjacent portions of the deck across the pier joints was selected over the more expensive deck and beam flange connection method. Since the live load capacity was sufficient, connecting only the deck served the purpose of eliminating the joints without the complexity and cost of connecting the beams together, which would have been required for full live load continuity. An initial link slab width of 3 feet was selected each side of the pier joint with adjustments as necessary to accommodate replacement of deck drains.

For the new link slabs, a portion of the existing deck was removed full-depth on either side of the joint. A second series of saw cuts were made to expose the existing reinforcement so that it could be lapped with the new link slab reinforcing steel. Concerns over debonding of the adjacent unreinforced concrete overlay prompted the link slab top longitudinal bars to be hooked for development purposes as opposed to doweling into the overlay. By hooking the top bars to ensure development, cracking within the link slab would be controlled and accommodated by strategically located sealed sawcut joints (Figure 5). Further, to facilitate rotation and minimize a rotational “hard point”, existing shear connectors were removed from the beams within the link slab and a pre-formed, closed-cell foam was applied to the beam and end diaphragm top flanges. It is important to note that the short spans for these bridges resulted in only nominal rotational demands on the link slabs that could be accommodated with the simple modifications described above. For longer bridges with greater rotational demands, significant analysis may be required to develop a properly performing link slab.
Abutment Conversion

Sizeable portions of the original cast-in-place concrete abutment, including the entire front wall behind the bearings and the vaulted deck slab, had to be removed to enable the conversion to semi-integral construction. Figure 6 is a section view reproduced from the design drawings showing the new abutment configuration. The existing fixed bearings at the abutments were replaced with new elastomeric bearings to accommodate longitudinal movements. A new concrete diaphragm beam was cast above the bearings to encase the ends of the steel beams, diaphragms, and associated connection hardware, as well as to provide a support for the new precast concrete abutment slabs replacing the demolished original deck. After positioning, these precast slabs were grouted in place and anchored to the abutment beam (Figure 7). The abutment beam was isolated from the original abutment structure to prevent any unintended contact or restraint against longitudinal movements. At the opposite end, the precast abutment slabs were installed upon neoprene bearing pads placed atop the remaining rear wall of the original vaulted abutment. A silicone sealant joint was installed between the precast abutment slabs and the adjacent rebuilt portion of the reinforced concrete approach slabs in order to accommodate the thermal movements of the bridge superstructure. Since the maximum expansion/contraction length for these bridges was around 70 feet, total thermal movements of 3/8 inch or less were expected at the abutment joints.

FIGURE 5. Section view through new link slab over pier joint

FIGURE 6. Section view through new semi-integral abutment
Accelerated Bridge Construction Practices

Due to the limited closure periods permitted by the ITRCC, several accelerated bridge construction practices were incorporated into the project design. It was anticipated that one-quarter of each bridge abutment conversion could be completed within a 4 1/2-day single-lane closure period. Other repairs could be done from underneath the bridge without lane closures, during shorter closure windows, or concurrently with the abutment conversion closures.

To allow traffic on the repaired bridge decks, sometimes within 24 hours of concrete placement, rapid setting concrete—both proprietary and ready-mixed—was specified for the contractor’s use. A traffic plate detail was developed to allow the contractor to open lanes even prior to achieving the minimum 24-hour compressive strength of 2500 psi.

Link slab staging details were proposed to provide continuity of the slab at the completion of the work. The contractor was required to complete removal and replacement of the bearings prior to commencing link slab and abutment conversions. To maintain some longitudinal restraint from traffic loads in the middle span prior to completing the link slab installation, tie plates were installed across the beams at the pier joints. These plates were later removed after link slab installation.

To facilitate the rapid removal and replacement of the vaulted abutment slab, a two-piece, precast concrete abutment slab was specified. A plan view is shown in Figure 8. The completed abutment slab was post-tensioned using a high strength rods tensioned to 50 kips to eliminate cracking at the grouted longitudinal joint and to provide for uniform live load distribution between pieces. However, each was designed to individually support traffic loads. A flashing detail was developed to cover the horizontal joint of the vaulted abutment side walls and precast abutment slab.
CONSTRUCTION

Work on the two mainline structures began in March 2013 and was substantially complete by May. The contractor utilized six 4 1/2-day closures to install the link slabs and complete the abutment conversions. Work on the link slabs was typically done first before the abutment conversions. The contractor’s typical procedure for the abutment conversions was to begin demolition of one quarter of the bridge starting early Monday morning with removal complete by the end of the day, form and install reinforcing steel on Tuesday, cast the diaphragm beam and approach pavement on Wednesday, set the precast abutment slab and cast the barrier wall on Thursday, and demobilize by Friday noon. A completed portion of the bridge deck is shown in Figure 9.

FIGURE 9. View of completed link slab (foreground) and abutment conversion (background)
CONCLUSIONS

Challenged to maintain numerous bridge structures approaching 60 years old, the ITRCC elected to pursue a joint elimination program to reduce future maintenance costs and preserve the existing bridge decks where possible. A field investigation determined that the two bridges discussed herein were suitable candidates to employ a novel approach to make the bridge decks continuous and convert the abutments to semi-integral construction. The approach utilized accelerated bridge construction techniques to accommodate the ITRCC’s lane closure policies to eliminate long-term lane closures on the ITR mainline during construction. The work was successfully completed in Spring 2013. Many of the design details have been subsequently used by others to repair similarly designed bridges for the ITRCC.

REFERENCES