An integral precast approach slab to bridge connection

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An integral precast approach slab to bridge connection

by:

Adam S. Faris

A thesis submitted to the graduate faculty
In partial fulfillment of the requirements for the degree of

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Program of Study Committee:
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ABSTRACT

The Iowa Department of Transportation has long recognized that approach slab pavements of integral abutment bridges are prone to settlement and cracking, which manifests itself as the “bump at the end of the bridge”. The bump is not a significant safety problem; rather it is an expensive maintenance issue. A commonly recommended solution is to integrally attach the approach slab to the bridge abutment, which moves the expansion joint typically found at the approach slab/abutment interface to a location further from the bridge where soil settlement is less of a concern and maintenance is easier. For existing structures the typical maintenance activities, such as adding wedges of asphalt to reduce the bump, are temporary fixes and do not address the underlying problem until the only remaining alternative is replacement of the approach slab. Finding ways rapidly make repairs, and thus decreasing the effects on traffic is critical. One promising solution for reducing construction durations, which the Iowa Department of Transportation has been testing and evaluating, is the use of precast concrete pavement and bridge elements.

For this study, two different approach slabs, one being precast concrete and the other being cast-in-place concrete, were integrally connected to twin parallel bridges on Iowa Highway 60. The primary objectives of this investigation were to evaluate: the viability of precast approach slabs, the approach slab performance, and the impacts the approach slabs have on the bridge.

The Iowa State University Bridge Engineering Center installed a monitoring system on both bridges and the approach slab systems. Several behavior facets were studied and monitored during the evaluation period including abutment movement, bridge girder strain changes, approach slab strain changes, approach slab joint displacements, post-tensioning strain, and abutment pile strain changes. The project scope also involved a literature review, a survey of Midwest Department of Transportation current practices, onsite observation of the fabrication and installation of the precast approach slab panels, and periodic visual inspection of the bridges.

From the onsite observations and the year-long monitoring the following general conclusions were made: (1) the integral connection appears to function well with no observed distress or relative movement of the approach slab and bridge; (2) the approach slab to the bridge connection appears to impact the bridge to a minor degree; (3) the two different approach slabs appear to impact the bridge differently; (4) the measured strains in the approach slabs indicate a force exists at the expansion joint which should be taken into consideration during the design stage; (5) the observed responses generally followed an annual cycle with shorter term cyclic patterns; (6) metal forms should be used to ensure a quality precast product; (7) larger access pockets should be used; and (8) finer granular material would allow easier and more precise adjustments of the panels.
CHAPTER 1  GENERAL INTRODUCTION

Introduction

The Iowa Department of Transportation (Iowa DOT) has recognized that approach slab pavements at integral abutment bridges are prone to settlement and cracking, which manifests itself as the “bump at the end of the bridge”. The bump is not a significant safety problem; rather it is an expensive maintenance issue. Further, public perception is negatively affected by the presence of the bump. The formation of the bump is typically attributed to settlement of backfill soil under the approach slab, deterioration of the corbel or paving notch, and poorly functioning expansion joints. Integral abutment (I-A) bridges are believed by many engineers to worsen the bump; although it is recognized that I-A bridges have many other highly desirable attributes. A commonly recommended solution is to attach the approach slab to the bridge abutment, which moves the expansion joint typically found at the approach slab/abutment interface to a location further from the bridge where soil settlement is less of a concern and maintenance is easier. Other states in the Midwest utilize this type of connection.

Damage to the approach slab and bridge abutment occurs as the bump worsens, due to the dynamic loads from vehicles, particularly heavy trucks. Typically maintenance activities, such as adding wedges of asphalt to reduce the bump, are just temporary fixes which do not address the underlying problem and damage tends to continue until the only alternative is replacement of the approach slab and/or paving notches. At least partial closure of the approach slab to traffic is required for the repair, which adds to negative public perception. This is particularly true in urban areas where lane closures should be minimized. Finding ways to increase the speed at which repairs can be made, and thus decreasing the effects on traffic is critical. One promising solution for reducing construction durations, which the Iowa DOT has been testing and evaluating, is the use of precast concrete pavement and bridge elements.

Two new twin parallel bridges on the new Iowa Highway 60 bypass of Sheldon, IA in O’Brien County were chosen as demonstration bridges for both the integral approach slab to bridge connection detail and the use of precast approach slab panels. The integral approach slab to abutment connection detail was implemented on both bridges while one bridge utilized a precast approach slab system and the other bridge a typical cast-in-place approach slab system. These were the first bridges in Iowa to tie the approach slab to an I-A abutment bridge. The Bridge Engineering Center at Iowa State University designed, implemented, and installed a monitoring system with the goal of determining the effect the connection had on the bridge and approach slabs and if that effect required the development of different design criteria.
Thesis Organization

A literature review and informal phone survey of other Midwest DOTs were conducted to find current practices and ideologies on integrally connecting the approach slab to the bridge abutment. The findings of the literature review and phone survey have been included in this thesis. The results and conclusions from the year long monitoring of the demonstration bridges have been divided and presented in two prospective journal papers. The first journal paper details the integral connection between both approach slabs and corresponding bridge abutments and will be submitted to the Bridge Engineering Journal. The second paper is focused on the precast approach slab used on the northbound bridge and will be submitted to the Precast Concrete Institute Journal (PCI Journal). The conclusions from the two papers are tied together at the end of this thesis.

Literature Review

I-A bridges, which are conceptually depicted in Figure 1.1, have become well known and widely used across the country. A study of current practices in the U.S. and Canada was performed by Kunin and Alampalli (2000). The authors reported the results of a 1996 survey of which 31 agencies responded to having experience with I-A bridges. Additionally, they found that by 1996 over 9,770 I-A bridges had been built. The popularity of I-A bridges stems from the many advantages they offer (Brena et. al. 2007; Burke 1993; Lawver et. al. 2000; Kunin and Alampalli 2000). Cost, both initial construction and long-term maintenance, is the biggest benefit derived from I-A designs due to the elimination of expansion joints and bearings. Generally I-A bridges experience less deterioration from de-icing chemicals and snowplows, decreased impact loads, improved ride quality, are simpler to construct, and have improved structural resistance to seismic events. Burke (1993) concludes that I-A bridges should be used whenever applicable because of the many advantages over the few disadvantages. One problem facing bridges nationwide is bump development at the end of the bridge. The bump problem appears to be a consistent problem with I-A bridges (Briaud et al. 1997).
In a literature review and survey of various state DOTs, Briaud et al (1997) summarized causes of the bump and offered potential solutions. According to the report “the bump develops when there is a differential settlement or movements between the bridge abutment and the pavement of the approach embankment.” This problem was estimated to impact 25% of the bridges in the country. Typically the bump is not a significant safety problem: rather it is an expensive maintenance issue. Three main causes for the bump can be taken from Briaud’s report. Figure 1.2 conceptually shows the causes which are summarized below:

1. Differential settlement between the top of the embankment and the abutment due to the different loads on the natural soil and compression of embankment soils, typically because of insufficient compaction.
2. Void development under the pavement due to erosion of embankment fill because of poor drainage.
3. Abutment displacement due to pavement growth, embankment slope instability, and temperature cycles on integral abutments.

While the above items seem to suggest that the problem is geotechnical and construction in nature, there is actually a structural issue present. Integral abutment bridges are called out as a distinct issue, with “many engineers responding to the survey believing the bump worsens with integral abutment bridges” (Briaud et. al 1997 pp. 25). Thermal cycles are a key behavior with I-A bridges since they do not have expansion joints and expand/contract with the thermal cycles. When I-A bridges expand, the fill material is compacted, creating a void that increases when the bridge contracts.
Schaefer and Koch (1992) also reported on the longitudinal movement of I-A bridges and the cyclic loading they impose on the backfill and foundation. As the temperature increases the superstructure and abutment move outward, toward the soil causing lateral earth pressures, and compacting the soil. As the temperature decreases, the bridge abutments move away from the compressed soil and a void forms (Figure 1.3). The creation of this void may lead to soil erosion that further increases the size of the void (White et al. 2005)
White et al. (2005 and 2007) investigated general bridge approach settlement in Iowa. At 25% of the 74 bridge sites (13 were I-A bridges) severe void development problems were observed. The authors indicate that void development commonly occurs within the first year after bridge approach pavement construction. Voids, and the erosion associated with void formation, lead to problems such as (1) exposing H-piles which potentially leads to accelerated corrosion and a reduction in capacity; (2) failure of slope protection; and (3) severe faulting in the approach slab caused by the loss of support. During observation of new I-A bridges under construction, White et al. found that poor construction practices may be another source of settlement of the approach pavement. The construction practices identified by the authors included poor approach pavement and paving notch construction, use of non-specified backfill material, and placing granular backfill in too thick of layers at the incorrect moisture for compaction. White et al. concluded that approach pavement systems were performing poorly because of poor backfill properties, inadequate subsurface drainage, and poor construction practices. They also reported that void development was more pronounced with I-A bridges.

In their 2005 report White et al. tested a variety of backfill soil types and geocomposite configurations. Some of the results were:

- Granular backfill, placed at bulking moisture content, undergoes 6% collapse compared to no collapse at 8% or higher moisture content.
- Granular backfill specified is highly erodible.
- Granular backfill can lead to large void development due to erodibility and compressibility at bulking moisture.
- Porous backfill does not experience collapse nor is it highly erodible.
- Porous backfill usage prevented approach settlement, void development, and increased drainage.

In a similar way, Briaud et al. (1997) gives several recommendations for best current practices associated with minimizing bridge approach ride issues. The recommendations are:

1. Make the bump a design issue with prevention as the goal.
2. Assign the design issue to an engineer.
3. Encourage teamwork and open-mindedness between geotechnical, structural, pavement, construction, and maintenance engineers.
4. Carry out proper settlement vs. time calculations.
5. Design an approach pavement slab for excessive settlement.
6. Provide for expansion/contraction between the structure and the approach roadway.
7. Design a proper drainage and erosion protection system.
8. Use and enforce proper specifications.
9. Choose knowledgeable inspectors, particularly on geotechnical aspects.
10. Perform inspections including joints, grade specifications, and drainage.

Of particular interest to this project is what Briaud et al. (1997) had to say about approach slabs (#5 in their best practice list). The report states that approach slabs are used by many states, with several states installing them on all bridges. Also reported was that “the use of reinforced approach slabs minimizes the bump or eliminates it all together,” and that “suggestions have been made to tie the approach slab to the abutment.”

In addition to recommending better backfill systems White et al. (2005) also recommended connecting the approach slab to either the abutment or the bridge deck. This eliminates the expansion joint at the bridge/approach slab interface. Both Briaud et al. (1997) and White et al. (2005 and 2007) made recommendations with regard to using approach slabs and the possibility of tying or integrally connecting them to the bridge as a way to minimize or eliminate the bump problem.

**Approach Slabs**

White et al. (2005) described approach slabs as being designed to be supported on the bridge abutment at one end and the fill or a sleeper slab (or beam) at the other. The purpose of the approach slab is to minimize differential settlement effects and to provide a transition from the pavement to the bridge deck. The level of performance of the approach slab is based upon many factors, including: (1) approach slab dimensions, (2) steel reinforcement, (3) the use of a sleeper slab, and (4) the type of connection between the approach slab and bridge.

Kunin and Alampalli (2000) found that there are two main approach slab to bridge connections. The first technique is to connect the slab reinforcement to the bridge through extension of the deck steel (see Figure 1.4). The second technique uses reinforcing steel to connect the slab to the corbel or abutment (see Figure 1.5). Another option to the two cited by Kunin and Alampalli is to have the approach slab rest on the paving notch of the abutment (see Figure 1.6). Hoppe (1999) reports that 71% of the state DOT's using I-A bridges use a mechanical connection between the approach slab and bridge.

A more recent survey conducted by Maruri and Petro (2005) found practices similar to those found by Kunin and Alampalli. Maruri and Petro suggest that standardization and guidelines would be beneficial for abutment/approach slab connections. They also found that 31% of the respondents use sleeper slabs, 26% do nothing but float the slab on the fill, and 30% do both.
Figure 1.4. Deck steel extension connection (standard Nevada detail)

Figure 1.5. Abutment steel connection (standard Ohio detail)

Figure 1.6. Abutment with no connection (standard Iowa detail)
Burke (1993) indicates that “full width approach slabs should be provided for most integral abutments and should be tied to the bridge to avoid being shoved off their seat by the horizontal cycle action of the bridge as it responds to daily temperature changes.” He also indicates with regards to approach slab to bridge connections that “approach slabs tied to bridges become part of the bridge, responding to moisture and temperature changes. They increase the overall structure length and require cycle control joints with greater ranges.” The cycle control joints are important because they relieve resistance pressures that are a result of the lengthening/shortening of the bridge. As the bridge moves, it is resisted by the approach slab in the form of a pressure. That pressure is distributed to both the slab and the bridge, but is a much greater problem for the pavement which has a smaller area. As a result, fracturing and buckling (i.e., blowouts) can occur in the approach pavement. Therefore cycle control joints must be designed and used. Burke also suggests another method to minimize the force required to move the approach slabs: “They should be cast on smooth, low-friction surfaces such as polyethylene or filter fabric.”

Similar to the above, Mistry (2005) recommends the following:

- Make installation of the approach slab a joint decision between the Bridge/Structures group and the Geotechnical group.
- Standardize the practice of using sleeper slabs, as cracking and settlement typically develops at the slab/pavement joint.
- Use well drained granular backfill to accommodate the expansion/contraction.
- Tie approach slabs to abutments with hinge type reinforcing.
- Provide layers of polyethylene sheets or fabric under approach slabs to minimize friction against horizontal movement.
- Limit skew to less than 30 degrees to minimize the magnitude and lateral eccentricity of longitudinal forces.

The above recommendations reinforce the emphasis to use proper backfill and friction reducing material under the approach slab. More importantly, Mistry's recommendations reinforce the importance of integrally connecting approach slabs to the bridge.

A report by Cai et al. (2005) noted the problem of the bump at the end of the bridge, repeating the causes previously discussed. They also recommended designing approach slabs to “span” the resulting voids. Designing the slabs as simply supported beams between the abutment and pavement ends is very conservative, and uneconomical. They also point out that the AASHTO code (AASHTO 2006) has no guidelines for designing approach slabs.

Due to the lack of guidelines, Cai et al. (2005) performed finite analysis on approach slabs loaded with a HS20 load while varying the amount of soil settlement. The resulting deflections and internal moments were recorded. Using the results of the finite element analysis and the parameters of the slab, formulas were developed to provide information for structural analysis and design of approach slabs for a given settlement. Cai et al. concluded that despite improving the approach slab design, the bump is still a function of settlement.
They noted that even if minimal settlement is allowed in the embankment soil through construction and geotechnical practices, there will always be a bump. A more rigid slab will have less deflection and change of slope but may increase soil pressures under the contact areas which are smaller due to spanning of any voids resulting in increasing faulting deflections.

There was very little literature found that investigates or discusses the effects that attaching the approach slabs to the I-A bridge has on the bridge itself. One report by Lawver et al. (2000) covers the instrumentation and study of an integral abutment bridge with tied approach pavement near Rochester, MN. The conclusion was that the bridge performed well during the reporting period, but that backfill material loss and void formation still occurred. There was no discussion directly on the effect the pavement may or may not have had.

Specific Practices

The reports on current practices, by Kunin and Alampalli (2000) and Maruri and Petro (2005), provide statistical summaries as to what many states do. They do not report many details and specifics on what individual states do, why they do it, or how they do it. In fact, there are only a few reports that go into detail on the specific practices.

The report by Yannotti, Alampalli, and White (2005) discussed the New York DOT experience with I-A bridges and presented specific practices. Of particular interest was the modification made to the approach slab to abutment connection after a 1996 study (similar to Figure 1.4). The older detail involved the extension of bridge deck steel horizontally into the approach slab. This detail was found to be unsatisfactory because the approach slab was unable to accommodate any settlement. This settlement typically caused transverse cracking in the bridge deck and transverse and longitudinal cracking of the approach slab. A new detail, shown in Figure 1.7, was developed using reinforcing bars at 45° into the bridge deck and the approach slab. This connection allows rotation of the slab by minimizing the moment capacity if the fill settles.

Harry White of the New York State DOT (NYSDOT) was contacted for further information. He added that the horizontal bar detail mentioned above provided negative moment capacity so that when the fill and slab settled, rotation was restrained leading to the cracking discussed above. He also indicated that the new detail (see Figure 1.7) is performing adequately and no notable problems have arisen. A requirement of NYSDOT and other states is the use of a polyethylene sheet under the full width of the slab to reduce sliding friction.
Since the New York report was one of only a few to discuss specific practices, bridge engineers at other DOTs were contacted for more information. With the assistance of the Iowa DOT nine other departments were contacted including those from Illinois, Kansas, Michigan, Minnesota, Missouri, Nebraska, North Dakota, South Dakota, and Wisconsin. Along with Iowa, these states make up the north central states. Engineers in each state were contacted first by email, followed by a phone conversation asking about specific practices regarding I-A bridges and approach slabs. The basic questions were:

- Do you typically connect the approach slab to the bridge? If so, how and why?
- How have the connections performed (any problems or good reports)?
- Has research or a study been performed?
- Is anything used beneath the slab to reduce friction?
- What is the backfill criterion in your state?

All the states, with the exception of Michigan participated. A summary of the practices of each state can be found at the end of this section in Table 1.1. Wisconsin was the only state that does not use a connection between the approach slab and the bridge. The contact, Lee Schuchardt, responded that the only change he would make would be to attach the slab to the abutment backwall with reinforcing bars because of the separation that happens between the abutment backwall and the approach slab.

Kevin Riechers of Illinois indicated that they have been building I-A bridges since the early 1980’s and began connecting the approach slab approximately five years after that. The typical detail used by Illinois is shown in Figure 1.8. This detail consists of #5 reinforcing
bars spaced every 12 in. that are extended horizontally from the bridge deck into the approach slab with 4 ft in the bridge deck and 6 ft in the approach slab. In addition, vertical #5 reinforcing bars are extended from the corbel into the approach slab every 12 in. The reason cited for connecting the slab and bridge was to keep the joint closed in order to keep water and debris out and the pavement moving with bridge. Transverse cracking of the slab was reported to be a problem. Mr. Riechers also reported that another problem is the settlement of the sleeper slab at the other end of the approach slab and that a new design is being considered. No research has been performed on approach slab to bridge connections. Also, nothing is apparently done to reduce surface friction under the approach slab except bond breaker between the slab and wing-walls of U-Back abutments. The soil is backfilled at the abutment with no compaction to avoid additional lateral earth pressures that may restrain thermal expansion of the bridge.

From Kansas, John Jones reported that approach slabs have been connected to the bridge for the last 12 years. The connection is made by extending #5 reinforcing bars horizontally from the bridge deck into the approach slab and ending in a standard hook (see Figure 1.9). The approach slab rests on a corbel at the bridge end and a sleeper slab at the other end, typically 13 ft away. The reason behind the connection was to remove the bump that formed at the end of the bridge. Though the bump was removed from the bridge end, it now appears between the slab and pavement. Mr. Jones reported that the connection has performed reasonably well and that public perception has been positive. Problems may arise if the sleeper slab settles, causing negative moments at the abutment. A solution to this is carefully mud-jacking the slab being mindful to avoid clogging the drain behind the abutment. No research as been performed and nothing is used to reduce friction. The backfill criteria used is the same as the road criteria (18 in. lifts at 90% compaction) with a strip drain installed behind the abutment.
Paul Rowekamp provided information on the practices in Minnesota. He reported that Minnesota has been building I-A bridges for approximately five to six years and connecting the approach slabs to the bridge for the last three years. The standard detail, shown in Figure 1.10, is to extend a #16 (metric, #5 U.S.) reinforcing bar diagonally from the abutment into the approach slab. This connection was implemented because of maintenance concerns pertaining to the opening of the joint between the slab and bridge. He explained that after the bridge has expanded to its limits, and begins to contract, the slab may not move with the bridge immediately because of friction with soil and lack of friction between the slab and the paving notch. Thus the joint opens slightly, filling with debris. The next season the same thing happens, filling the joint with more debris. The slab now has less to rest on, and water can now flow in and beneath the slab. As the slab approaches the edge of the paving seat, it may eventually fall completely off. Mr. Rowekamp reported that the initial connection design used an 8 ft horizontal bar extending 4 ft each way into the slab and bridge deck. Transverse cracking across the entire approach slab appeared approximately where the horizontal bar ended, possibly caused by rotation of the slab being restrained. Two years ago a change was made to the current detail, and no problems have been reported thus far. No research has been performed on the connection. Minnesota standard details do not call for any friction reducing material. Backfill of the abutment is specified as modified select granular material (having no fines) and is installed in typical lifts and compacted.
David Straatmann, with the Missouri DOT, indicated that connecting the approach slab to the bridge has been standard practice for some time. The standard connection method, shown in Figure 1.11., is made by extending #5 reinforcing bars, spaced at 12 in., horizontally between the bridge deck and approach slab. Two layers of polyethylene sheeting are used between the approach slab and construction base. No information was given in regards to the reason why this connection is used, performance of this connection, research performed, and backfill criteria.

In Nebraska, according to Scott Milliken, approach slabs have been used for the last 15 years, with connecting the slab to the bridge being the standard practice for at least the last 10 years. The standard connection method, shown in Figure 1.12., is made by #6 reinforcing bars that extend vertically from the abutment, then bent at 45° into the approach slab. Nebraska refers to the approach slab as an approach section, which rests on a grade beam.
supported by piles at the end opposite the bridge. From the grade beam to the pavement, another transition section, called the pavement section is used. According to Mr. Milliken, the reason for the connection was to eliminate, or at the least, move the bump from the end of the bridge to a location that is more easily maintained. This methodology also eliminated water from infiltrating the bearing of the bridge. A problem arising from the approach slabs was settlement of the sleeper slabs in the original design, leading to the use of grade beams as described above. Recently, hairline cracks, perpendicular to the grade beams on bridges with severe skews, were discovered. A top mat of steel was added in the approach slab, but no feedback was yet available. Overall, management is pleased with the performance thus far. No research has been performed on the approach slabs and connection. There is nothing done to reduce the friction between the slab and the ground. Fill behind the abutment is considered only necessary until the concrete in the approach section reaches strength, at which time it acts like a bridge between the abutment and grade beam. Granular backfill is used, with drainage provided by drainage fabric. The material is installed in lifts and compacted with smaller equipment to avoid damaging the wing-walls.

![Figure 1.12. Typical Nebraska detail](image)

According to Tim Schwagler of the North Dakota DOT, for approximately the last five years the practice in North Dakota has been to connect the approach slab to the bridge. This is accomplished by mechanically splicing a horizontal extension of #5 reinforcement from the bridge deck to the approach slab every 12 in. with joint filler (polystyrene), as shown in Figure 1.13. Two different types of approach slabs are used. On newer sites and newer embankments the far end of the approach slab is supported on piles. When approach slabs are used on older sites where settlement is assumed to have already occurred in the embankment soil, the far end of the approach slab rests on the base course. This connection was implemented to improve joint performance between the approach slab and bridge. One-inch joints were installed with filler and joint sealant. The North Dakota DOT found that the joints were opening and tearing the sealant. The connected joints have
performed very well and no adjustments have been made. No research has been performed, and there is nothing done to reduce friction between the slab and the ground. When the abutments are backfilled a trench at the bottom 2 ft – 6 in. deep is filled with rock wrapped in fabric with a drain pipe. Granular material ND Class 3 or 5 is then placed in 6 in. lifts and compacted.

According to Steve Johnson of the South Dakota DOT, the standard practice is to almost always connect the approach slab to the bridge deck on I-A bridges. This has been the practice for approximately the last 25 years. The connection is made by extending a #7 reinforcing bar that is embedded horizontally 2 ft into the bridge deck into the approach slab for 2 ft every 9 in. as shown in Figure 1.14. A mechanical splice is used to make construction easier. After backfilling of the abutment is complete, the horizontal reinforcement is spliced. The connection is used to keep water from flowing into the backfill and to provide a smoother transition while driving, because the “bump” is at least moved to the end of the approach slab. According to Mr. Johnson, the connection has performed relatively well over the years. One change was made after transverse cracking was noticed 4 to 5 ft. from the bridge. It was determined that the reinforcement was “too high” in the slab, so the design was changed to have the connection steel deeper in the slab. The only other problem reported is that the far end of the approach slab sometimes settles. No research has been performed on the connection. Plastic sheeting is required beneath the approach slab, not to reduce sliding friction, but to create a mud-jack barrier, so that mud is not lost into the voids of the base course, if it must be performed. When the abutment is backfilled, drains are installed along the backside of the abutment. The first 3 ft from the abutment is free draining granular material. After that typical fill (unspecified) is brought up in 8 to 12 in. lifts and compacted as best as possible.

![Figure 1.13. Typical North Dakota detail](image-url)
Table 1.1. Summary of DOT responses

<table>
<thead>
<tr>
<th>State</th>
<th>Connection</th>
<th>Performance</th>
<th>Research</th>
<th>Friction Reduction</th>
<th>Backfill Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>Illinois</td>
<td>Yes - Horizontal</td>
<td>Transverse cracking problem</td>
<td>No</td>
<td>No</td>
<td>Uncompacted</td>
</tr>
<tr>
<td>Kansas</td>
<td>Yes - Horizontal</td>
<td>Reasonably well</td>
<td>No</td>
<td>No</td>
<td>18 in. lifts, 90%</td>
</tr>
<tr>
<td>Minnesota</td>
<td>Yes - Diagonal</td>
<td>No problems reported</td>
<td>No</td>
<td>No</td>
<td>Modified select granular material compacted in lifts</td>
</tr>
<tr>
<td>Missouri</td>
<td>Yes - Horizontal</td>
<td>N/A</td>
<td>No</td>
<td>Yes</td>
<td>N/A</td>
</tr>
<tr>
<td>Nebraska</td>
<td>Yes - Diagonal</td>
<td>Management is pleased</td>
<td>No</td>
<td>No</td>
<td>Compacted granular material</td>
</tr>
<tr>
<td>North Dakota</td>
<td>Yes - Horizontal</td>
<td>Very well</td>
<td>No</td>
<td>No</td>
<td>Granular material compacted in 6 in. lifts</td>
</tr>
<tr>
<td>South Dakota</td>
<td>Yes - Horizontal</td>
<td>Pretty well</td>
<td>No</td>
<td>No</td>
<td>Granular fill for drainage, then typical fill compacted in 8 to 12 in. lifts</td>
</tr>
<tr>
<td>Wisconsin</td>
<td>No</td>
<td>N/A</td>
<td>No</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

N/A = Not Applicable

References


Yannotti, A. P., Alampalli, S., White, H. L. (2005). “New York State Department of Transportation’s Experience with Integral Abutment Bridges.” FHWA Conference: Integral Abutment and Jointless Bridges (IAJB 2005), FHWA, West Virginia University, March 16-18, 2005, pp. 41-49
CHAPTER 2 INTEGRAL BRIDGE ABUTMENT-TO-APPROACH SLAB CONNECTION

A paper to be submitted to The Bridge Engineering Journal

Adam Faris, Brent Phares, Lowell Greimann, Jake Bigelow, Dean Bierwagen

Abstract

This paper presents the findings of the investigation of two approach slabs (a cast-in-place slab and a precast panel slab) integrally connected to two parallel bridges. The goal of using the integral connection is to eliminate the “bump at the end of the bridge”. The investigation’s primary objectives were to evaluate the approach slab behavior and the impact of the approach slabs on the bridge. A long-term structural monitoring system consisting of various vibrating wire transducers were installed. From the year-long monitoring the following general conclusions were made: (1) the integral connection appears to function well with no observed distress or relative movement between the approach slab and bridge; (2) the approach slab to the bridge connection appears to impact bridge behavior; (3) the two different approach slabs appear to impact the bridge differently; (4) the measured strains in the approach slabs indicate a force exists at the expansion joint which should be taken into consideration during the design stage; (5) the observed responses generally followed an annual cycle with short term cyclic patterns also apparent.

Introduction

The Iowa Department of Transportation (Iowa DOT) has recognized that approach slab pavements of integral abutment (I-A) bridges are prone to settlement and cracking, which manifests itself as the “bump at the end of the bridge.” The bump is not a significant safety problem; rather it is an expensive maintenance issue. Further, public perception is negatively affected by the presence of the bump.

The formation of the bump is typically attributed to settlement of backfill soil under the approach slab, deterioration of the corbel or paving notch, or poorly functioning expansion joints at the bridge abutment. Many studies have been performed on approach slab settling and the formation of the bump (Briaud et al. 1997; Schaefer and Koch 1992; White et al. 2005 and 2007) resulting in suggested backfill and subgrade material types as well as improved construction practices.

Integral abutment bridges, which are becoming more popular, are believed by many engineers to worsen the bump; although popularity in I-A bridges stems from the many desirable attributes such as lower initial construction costs and fewer long-term maintenance costs (Brena et. al. 2007; Burke 1993; Lawver et. al. 2000; Kunin and Alampalli 2000).
Burke (1993) concluded that I-A bridges should be used whenever applicable because of the many advantages over the few disadvantages. The bump problem, however, appears to be a consistent problem with I-A bridges (Briaud et al. 1997). One possible way to improve long-term performance is to integrally attach the approach slab to the bridge abutment, which moves the expansion joint typically found at the approach slab/abutment interface to a location further from the bridge where soil settlement is less of a concern and maintenance may be easier.

In an effort to eliminate the bump problem, the Iowa DOT designed a new detail for the approach slab to abutment interface. Deviating from the previous standard of casting the approach slab on a corbel and installing an expansion joint between the approach slab and bridge deck, the new detail integrally connects the approach slab to the abutment with steel dowels. As a result, the expansion joint is moved to the other end of the approach slab. A team from Iowa State University designed and implemented a monitoring system for the first project in Iowa to use the new connection. The objective of this monitoring system implementation was (1) to determine the effect the connection had on the bridge and approach slab, and (2) to determine whether that effect required the development of different design criteria for the bridge structure and/or the approach slab.

**Bridge Description and Instrumentation**

Two new side-by-side three span bridges constructed in 2006 on the new Iowa Highway 60 bypass of Sheldon, IA in O’Brien County were selected as test bridges for evaluation of the previously mentioned approach slab to bridge connection detail. The same connection detail was implemented on both bridges with the only difference being the type and length of approach slabs. These are the first bridges in Iowa to tie the approach slab to an I-A bridge. The northbound bridge utilized an approximately 23.5 m (77 ft) long precast approach slab system (Figure 2.1 and Figure 2.2) that meets the mainline pavement at an Iowa DOT standard EF joint. The EF joint is an expansion joint filled with flexible foam and contains 457 mm (18 in.) long 38 mm (1½ in.) diameter dowels spaced every 305 mm (12 in.) at pavement mid-depth and provides up to 102 mm (4 in.) of movement. The dowels were inserted into sleeves cast into the ends of the precast panels with greased tubes. The southbound bridge utilized an approximately 9.1 m (30 ft) long cast-in-place approach slab system (Figure 2.3) ending at the mainline pavement with an Iowa DOT standard CF joint. Unlike the EF joint, the CF joint does not have dowels and uses tire buffings instead of flexible foam. Both expansion joints were capped with 38 mm (1½ in.) joint sealer. All other aspects of the bridges were identical: 92.4 m (303 ft) x 12.2 m (40 ft) three-span-continuous, right-hand-ahead 30 degree skew, supported by HP10x57 piles (nine at the abutments and seventeen at the piers) with the webs oriented in the plane of the skew. Details of the cast-in-place slab reinforcing are shown in Figure 2.4. Details of the precast approach slab panels are shown in Figure 2.5 and Figure 2.6
The research team instrumented the south approach slab, abutments, and south end span of both bridges in order to determine the performance of the approach slab, the effects of attaching the slabs on the bridge, and the possible range of forces to consider when designing connected approach slabs (in both the bridge and slab). A wide variety of sensors were installed on and within the bridge and the approach pavement to monitor: temperature, bridge abutment movement (translation and rotation), bridge girder strain changes, approach slab strain changes, post-tensioning strand losses, approach slab joint relative displacement, and bridge abutment pile strain changes. Data were collected every hour for approximately one year (April 2007 to April 2008).

Figure 2.1. Plan view of precast approach slab (northbound bridge)

Figure 2.2. Connection detail for the precast approach slab to abutment
Figure 2.3. Plan view of cast-in-place approach slab (southbound bridge)

Figure 2.4. Elevation view of cast-in-place approach slab with connection detail

Figure 2.5. Precast panel detail along longitudinal edge
Bridge Abutment Movement

The data from the abutment displacement gauges and tiltmeters were corrected to account for the temperature changes of the instruments and added to estimate the longitudinal displacement of the abutment at mid-depth of the approach slab. Measurements were taken at both the east and west ends of the abutments. Positive readings were movements to the south, or away from the center of the bridge. The displacement ranges are presented in Table 2.1. The average abutment displacement range between the east and west side was 6.4 mm (0.25 in.) greater for the southbound bridge than the northbound bridge. This difference is likely due to the difference in length of the approach slabs and the subsequent resistance to movement and/or the efficiency of the expansion joints and the similar lateral resistance.

Transverse abutment displacements were measured at one end of each abutment, at the west end of the northbound bridge abutment and the east end of the southbound bridge abutment. The northbound bridge had a transverse displacement range of 24.1 mm (0.95 in.), which is greater than the southbound bridge range of 14 mm (0.55 in) (Table 2.1). Though not shown here, the transverse displacement of the northbound bridge abutment was not the same as the southbound bridge abutment over time. Specifically, the northbound abutment displaced to the east as the temperature decreased (July to January) while the southbound abutment displacement was relatively constant from July to October before displacement to the west occurred. The reason for the transverse movement difference between bridges could not be resolved definitively. It is thought, however, that the bridge skew may have had some impact on the difference in measurements (due to the fact that the measurements were not taken on the same side of the bridge, relative to the skew).
Table 2.1. Abutment Displacements

<table>
<thead>
<tr>
<th>Action</th>
<th>Northbound</th>
<th>Southbound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abutment Longitudinal Displacement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>West End Range</td>
<td>22.9 mm (0.90 in)</td>
<td>20.3 mm (0.80 in)</td>
</tr>
<tr>
<td>East End Range</td>
<td>22.9 mm (0.90 in)</td>
<td>34.3 mm (1.35 in)</td>
</tr>
<tr>
<td>Average Range</td>
<td>22.9 mm (0.90 in)</td>
<td>26.7 mm (1.05 in)</td>
</tr>
<tr>
<td>Abutment Transverse Displacement</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse Range</td>
<td>24.1 mm (0.95 in)</td>
<td>14 mm (0.55 in)</td>
</tr>
</tbody>
</table>

The measured longitudinal abutment displacement at the centerline of the northbound and southbound bridge abutments are shown, along with the expected displacement, over time in Figure 2.7 and Figure 2.8, respectively. The expected displacement, calculated by Equation 1, is the displacement that would have occurred if the bridge expanded unrestrained from the center of the bridge.

\[ \Delta = \alpha \cdot \Delta T \cdot \frac{L}{2} \]  

\[ \alpha = \text{Coefficient of thermal expansion for the bridge girders and deck} = 9.0 \times 10^{-6} \text{mm/mm/°C} \]  
\[ \Delta T = \text{Measured change in bridge temperature} = 56.7 \degree \text{C (134}\degree \text{ F)} \]  
\[ L = \text{Length of the bridge} = 92.4 \text{ m (303 ft)} \]  

The bridge temperatures used throughout this work are the average of the temperature readings collected at the girder strain gauge locations (note that these temperatures compared well to the air temperatures observed in Sheldon Iowa as recorded on “The Weather Channel” website). The coefficient of thermal expansion for concrete (\( \alpha \)) used in the expected displacement calculations was estimated by the method presented by Abendroth and Greimann (Abendroth and Greimann 2005), which is based on the specific concrete mix ingredients and proportions. In this work an \( \alpha \) equal to 9.0x10^{-6} \text{mm/mm/°C (5.0x10^{-6} in./in./°F)} \) was used. The expected displacement range of 30.5 mm (1.2 in.) is greater than the measured displacement range of either bridge, which again maybe due to the difference in slab lengths or expansion joint performance. Interestingly, the trend of the measured displacements does not chronologically match the expected displacements. The reason for the difference is probably due to a behavior termed friction ratcheting, which is discussed subsequently. During design the Iowa DOT assumed a temperature range of
37.8°C (100°F) and an α equal to $10.8 \times 10^{-6}$ mm/mm/°C ($6.0 \times 10^{-6}$ in./in./°F) which is equal to a movement of 28 mm (1.1 in.).

When the measured abutment displacement is plotted versus the bridge temperature, the relationship between abutment movement to changing temperature becomes apparent. Figure 2.9 presents this behavior for the northbound bridge (note that similar results observed for the southbound bridge). Different colors were used for different seasons to show the passage of time. As the bridge cooled, it contracted (the abutments moved inwards) illustrated by line A. During the annual thermal cycle, the abutment displacement versus temperature plot formed a large loop from Spring 07 to Spring 08, when the abutment returned to its approximate beginning position. Smaller loops, which may be due to friction ratcheting (discussed subsequently), can be seen. These smaller, short duration loops are highlighted by the parallel lines B and C.

Figure 2.7. Average and expected abutment displacement of the northbound bridge

Figure 2.8. Average and expected abutment displacement of the southbound bridge
Bridge Superstructure Behavior

The behavior of the superstructure was also monitored. Strain gauges were installed on the girders of the south span of both bridges. Each bridge has seven girders with three of the seven girders (the second, fourth, and sixth) instrumented at three cross-sections along the length of each girder: the ends and the mid-span of the girder with gauges placed on the top and bottom flange at each cross-section. From the strain and temperature data the temperature induced moment and axial force in the girders were calculated. Table 2.2 presents the resulting average girder moments at mid-span and the axial force for each bridge. The range of the northbound bridge moment was 8% larger than the southbound bridge moment range. The total annual change in thermal induced mid-span moment, of 881 kN-m (650 kip-ft), is only 12% of the total moment capacity, 7114 kN-m (5245 kip-ft), of the LDX90 girders.

While the ranges of mid-span moments only differed by 8% between the bridges, the behavior of each bridge over time was very different. The northbound bridge experienced a negative moment (compression on the bottom) of -678 kN-m (-500 kip-ft) at the same time that the southbound bridge experienced a positive moment of 745.7 kN-m (550 kip-ft) as illustrated in the trends of the moment versus temperature plots in Figure 2.10 and Figure 2.11. (Note that the solid trend line highlights the annual trend while the dashed trend lines highlight short term trends). The reason for the radical difference in measured behavior was never resolved with any certainty.
Table 2.2. Girder Forces

<table>
<thead>
<tr>
<th>Action</th>
<th>Northbound</th>
<th>Southbound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder Midspan Moment</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average Min</td>
<td>-678 kN-m</td>
<td>-68 kN-m</td>
</tr>
<tr>
<td></td>
<td>((-500 \text{kip*ft}))</td>
<td>((-50 \text{kip*ft}))</td>
</tr>
<tr>
<td>Average Max</td>
<td>203 kN-m</td>
<td>746 kN-m</td>
</tr>
<tr>
<td></td>
<td>((150 \text{kip*ft}))</td>
<td>((550 \text{kip*ft}))</td>
</tr>
<tr>
<td>Average Range</td>
<td>881 kN-m</td>
<td>814 kN-m</td>
</tr>
<tr>
<td></td>
<td>((650 \text{kip*ft}))</td>
<td>((600 \text{kip*ft}))</td>
</tr>
<tr>
<td>Average Girder Axial Load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range</td>
<td>3114 kN</td>
<td>2891 kN</td>
</tr>
<tr>
<td></td>
<td>((700 \text{kips}))</td>
<td>((650 \text{kips}))</td>
</tr>
</tbody>
</table>

Figure 2.10. Northbound bridge average mid-span moment versus bridge temperature
Figure 2.11. Southbound bridge average mid-span moment versus bridge temperature

Bridge Substructure Behavior

Strain gauges were installed on three piles under the south abutment of each bridge. The gauges, four on each pile, were installed on the edges of the pile flanges a few feet below the bottom of the abutment. Intentions were to observed and compare the pile axial and flexural strains of each bridge. Unfortunately due to instrumentation failure, only one pile was able to be observed and no notable conclusions could be made.

Approach Slab Behavior

As mentioned previously this was the first project in Iowa to use an integral approach slab to bridge connection. This was also the first project in Iowa to use a precast approach slab. Obviously, the performance and behaviors of the approach slabs were of interest for both reasons. One initial concern was that during the annual thermal cycle the joints between the approach slab and bridge and between the individual precast panel joints would open due to the push/pull of the bridge thermal expansion/contraction. Crackmeters were installed across the five joints (abutment to slab, Panel 1 to 2, Panel 2 to 3, Panel 3 to 4, and the expansion joint between Panel 4 and the mainline pavement (refer to Figure 2.1 for panel locations)) to monitor the behavior of the joints. In short, the results (Figure 2.12) indicate that most joints behaved as designed. Specifically, negligible relative movement was observed between the panels and between the bridge abutment and panels, while the relative movement at the expansion joint was approximately 25.4 mm (1 in.) The results were similar to the southbound bridge with the cast-in-place approach slab. The results are also summarized in Table 2.3.
Table 2.3. Approach Slab Results

<table>
<thead>
<tr>
<th>Action</th>
<th>Northbound</th>
<th>Southbound</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Slab Load Strain</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range</td>
<td>120 µε</td>
<td>105 µε</td>
</tr>
<tr>
<td><strong>Slab Forces</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Range of Force/Stress</td>
<td>10.2 kN / 4MPa</td>
<td>8.5 kN / 3.6 MPa</td>
</tr>
<tr>
<td></td>
<td>(2300 kips / 580 psi)</td>
<td>(1900 kips / 520 psi)</td>
</tr>
<tr>
<td>Friction Range</td>
<td>-58.5 kN/m to 58.5 kN/m</td>
<td>Information Not Available</td>
</tr>
<tr>
<td></td>
<td>(-4 to 4 kips/ft)</td>
<td></td>
</tr>
<tr>
<td><strong>Joint Movement</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Abutment Joint</td>
<td>&lt;1 mm (&lt; 0.03 in)</td>
<td>&lt;1 mm (&lt; 0.03 in)</td>
</tr>
<tr>
<td>Expansion Joint</td>
<td>23 mm (0.9 in)</td>
<td>28 mm (1.1 in)</td>
</tr>
</tbody>
</table>

Figure 2.12. Northbound bridge precast approach slab joint movements

As the approach slab expands and is pushed by the bridge during thermal expansion, the movement is expected to be resisted by friction between the approach slab and the subgrade. The expected result is that the longer the approach slab is, the larger the total friction force. Polyethylene sheets were installed under all the approach slabs in an effort to reduce the friction between the slab and the subgrade. Strain gauges were embedded at the
middle of each precast panel with the intention of observing the forces in the slab and for estimating the frictional components of the force. Figure 2.13 shows the total force in the precast slab with respect to location along the length of the slab for typical warm and cold days. The force in the slab was found by multiplying the average measured strain by the modulus of elasticity and then by the cross sectional area of the concrete. Each data point is the average of the four gauges at that longitudinal location as measured from the expansion joint.

There is a slope to the force graph as indicated by the least squares straight shown in Figure 2.13. Figure 2.14 shows a free body diagram indicating a positive friction force, f, and a conceptual extrapolation of the best fit line to determine the forces at the expansion joint and abutment. Figure 2.15 shows the friction per unit slab length, f, of the precast approach slab on the base material during the monitoring period. The maximum friction ranged from 117 kN/m to -117 kN/m (8 to -8 kip/ft ) (coefficient of friction equal to the friction force/slab weight of 1.9) however, the majority of the friction ranged from -58.5 kN/m to 58.5 kN/m (4 to -4 kip/ft ) (coefficient of friction of 0.95), which corresponds to a friction angle of approximately 43\(^\circ\). This is slightly higher than the upper limit presented by Das (1998) of 38\(^\circ\) for friction between sand and concrete. This suggests that the polyethylene sheeting did not significantly reduce the friction between the approach slab and subgrade material. It may be that, despite the sheeting under the precast slab, the grout formed a rough surface, like cleats, due to voids in the base course.

The force at the expansion joint and abutment was calculated using the best fit procedure illustrated in Figure 2.14. The precast slab had slightly larger compression forces at the expansion joint than at the abutment, except during the spring and latter part of winter. If the expansion joint were functioning as intended, the measured force at the expansion joint would be zero, which it is not (see Figure 2.13). Instead there is a substantial change in the slab force during the year (e.g., the difference between a cold day in January and a warm day in September shown in Figure 2.13). The total axial stress in the slab had a range of 4.0 MPa (580 psi) (Table 2.3) which is approximately 8% of the 48.3 MPa (7000 psi) design compressive strength. The shorter cast-in-place slab had a lower axial stress range of 3.6 (520 psi). The difference in axial stress may be due to the different approach slab lengths or the behavior of the different expansion joints or a combination of both.
Figure 2.13. Warm and cold precast approach slab load force with respect to location from the expansion joint

Figure 2.14. Free body diagram of friction force in slab
Figure 2.15. Bottom of slab friction over time – northbound bridge

The longitudinal post-tension strands of the precast slab were also observed. A loss of 4.9 kN (1.1 kips) of post-tension force occurred during the study, which is equal to about 2.5% of the initial post tensioning force. The transverse post-tensioning strands had a loss of approximately 4.45 kN (1 kip) of post-tension force. It should be noted that the observation period began seven months after installation and post-tensioning of the precast panels.

Friction Ratcheting

The friction and the resistance occurring at the expansion joint may explain other trends noticed in the bridge data. Hassiotis observed “strain-ratcheting” (Hassiotis, 2005) of the soil behind the abutment. In the subject situation, with a long approach slab that is attached directly to the abutment, a phenomena that produces a similar ratcheting, “friction ratcheting”, was observed. As the bridge expands, it pushes against the approach slab. Initially, the friction beneath the approach slab and forces at the approach slab/pavement expansion joint resist this motion. If the bridge expansion is sufficiently large, the static friction forces will eventually be overcome and the slab will slide. As the bridge cools, the bridge will eventually slide the slab in the opposite direction. The short term trends of small “loops” of data were observed in many of the results plots like Figure 2.9, Figure 2.10, and Figure 2.11. For example, as illustrated Figure 2.9 during the summer and fall (July to December) the relationship between abutment displacement and temperature is generally positive (see Line A). However, short term cycles with a negative slope were also evident (Line B). These cycles occur for short term temperature variations and are probably due to
the friction-ratcheting described above. During a short term cycle, the static frictional resistance under the slab may not be overcome and the bridge is pushed back and forth by the expanding and contracting slab. Eventually, however, as the temperatures continue their seasonal trend, the slab friction is overcome and the slab slides to a new position where again, for a period of time, the bridge and slab cycle without sliding. During the winter and spring period, the short term cycles with a negative slope are evident (Line C).

**Visual Inspection**

In addition to results from the instrumentation, site visits were periodically made to inspect the bridge site. During these visits basic visual inspections of the bridges and approach slabs were performed. No issues were found until the spring of 2008 (at the end of the first observed thermal cycle). At that time two hairline transverse cracks were observed in the south approach slabs, one on each bridge. One crack was found in the middle of Panel 2A of the precast approach slab. That tight crack began at the east shoulder and continued across the panel ending at the grouted longitudinal joint. In the doubly reinforced section (refer to Figure 2.3) of the cast-in-place southbound approach slab, along the east shoulder, a 737 mm (29 in.) long transverse crack was found 4.6 m (15 ft) from the approach slab to bridge joint.

At the north end of the southbound bridge, several cracks were noticed in the cast-in-place approach slab. Many tight, 51-76 mm (2-3 in.) long transverse cracks were visible along both shoulders of the non-reinforced section (refer to Figure 2.3 – the north end is mirror of the south end). A 1067 mm (42 in.) long transverse crack was found that started on the east shoulder approximately 711 mm (28 in.) from the north construction joint in the singly reinforced approach slab section. Despite the lack of longitudinal post tensioning, the cracks remain closed as can be seen in Figure 2.16.

A void was found under the approach slab at the south end of the southbound bridge on the west edge where the approach slab and concrete barrier rail meet. The void, shown in Figure 2.17, was 660 mm (26 in.) wide (152mm (6 in.) next to slab, 508 mm (20 in.) under the slab), 330mm (13 in.) long, and 559 mm (22 in.) deep. Loss of soil under the approach slab near the bridge abutment has been repeatedly cited (Briaud et al. 1997; Schaefer and Koch 1992; White et al. 2005 and 2007) as being a contributing factor to the bump at the end of the bridge. One attractive design aspect of the integral approach slab/bridge connection is that it was thought that it would reduce the amount of water infiltrating the subgrade by keeping the joint closed tightly instead of continuous expansion/contracting with debris entering the voids and relative movement between the slab and abutment. The loss of soil was only observed at the southbound bridge with the cast-in-place approach slab. The use of the precast approach slab required under-slab grouting which may have reduced water infiltration and soil loss beneath the slab.
Figure 2.16. Transverse cracking of the doubly reinforced approach slab at the south end starting at the east shoulder

Figure 2.17. Void under the west edge of the approach slab at the bridge abutment
Conclusions

The use of an integral approach slab to bridge connection did not cause any readily apparent damage to the bridges. No visible signs of distress were observed during any visit to the site, and the measured strains and calculated forces were not excessive enough to raise concern. The smaller movement range of the northbound bridge abutment may have been due to the longer approach slab used or the improperly functioning EF joint, both of which could have created end restraint leading to less movement. Regardless, the integral connection still allowed the bridges to move with the thermal cycle, with movements similar to those expected.

The difference in approach slab length and expansion joint types may have caused some differences between the behaviors of the bridge superstructures; however, the ranges of the measured forces were within 10% of each other and were only 12% of the girder capacity. Since the differences in the superstructure behavior could not be explained, no recommendations for different practices can be made at this time. A monitoring system should be implemented if the total approach slab exceeds 24.3 m (80 ft) in length, since the length of the approach slab may be the reason for the differences.

The joint between the approach slab and bridge was the same for both bridges. Both the northbound and southbound integral connection functioned properly during the year of observation by not allowing the slab or bridge to move independently while also keeping the joint closed. Based on this yearlong study and observation of the bridge, this joint design is adequate to integrally connect the approach slab and bridge.

The approach slab force was expected to decrease from the joint at the abutment to the EF expansion joint at the mainline pavement as the bridge movements were resisted by friction under the slab. At the EF joint the slab force was expected to be zero. Instead the strain was found to have a “small” change along the entire length of the slab, with a “notable” force present at end of the slab. It appears that the EF joint is not functioning as designed which is most likely due to misalignment during installation. Since the EF joint is not functioning as designed, the slab cannot freely expand and contract and stresses build up in the slab. There is no evidence that the stress at the EF joint is large enough to cause significant problems, though in the future specific attention should be paid to ensure the joint is installed correctly and functions as designed.

No definite conclusions could be drawn as to whether the difference in results between the bridges was caused by the different approach slab type, the different slab lengths, or from the performance of the different expansion joints. From a design stand point the approach slab to bridge connection appears structurally adequate. However, a serviceability study should be performed on the bridge to determine if the connection between the approach slab and the bridge reduced or eliminated the “bump at the end of the bridge”.
References


CHAPTER 3  IOWA’S EXPERIENCE WITH PRECAST PAVEMENT APPROACH SLAB

A paper to be submitted to The PCI Journal

Adam Faris, David Merritt, Brent Phares, Dean Bierwagen

Synopsis

This paper summarizes a pilot project undertaken by the Iowa Department of Transportation to evaluate the use of precast prestressed bridge approach slabs that are tied to integral abutment bridges. The precast approach pavement system is intended for use in either new construction or repair applications and can be installed in single lane width panels to permit staged construction under traffic. This project was successful in demonstrating the viability of precast concrete panels for approach slab construction and provided the Iowa DOT information on ways to improve cost and quality of the precast pavement, improvements for installation practices, and information on design parameters based on onsite observations and the results from a yearlong structural health monitoring system involving a variety of vibrating wire strain and displacement gauges.

Introduction

The Iowa Department of Transportation (Iowa DOT) has recognized that bridge approach slabs are prone to settlement and cracking, which manifests itself as the “bump at the end of the bridge.” The bump is generally not a significant safety problem, but is an expensive maintenance issue, and public perception is negatively affected by the presence of the bump. The formation of the bump is typically attributed to settlement or erosion of backfill soil under the approach slab, deterioration of the corbel or paving notch, and poorly functioning expansion joints at the bridge abutment. Many studies have examined approach slab settlement and the formation of the bump (Briaud et al. 1997; Schaefer and Koch 1992; White et al. 2005 and 2007) resulting in suggested improvements for embankment materials and construction practices. While these studies have found various reasons for the bump, and have provided suggestions for preventing the bump from forming, methods are still needed to efficiently repair existing approach slabs with these problems.

Damage to the approach slab and bridge abutment occurs as the bump worsens, due to the dynamic loads from vehicles, particularly heavy trucks. Typically maintenance activities, such as adding wedges of asphalt to reduce the bump, are just temporary fixes which do not address the underlying problem and damage tends to continue until the only alternative is replacement of the approach slab and/or paving notches. At least partial closure of the approach slab to traffic is required for the repair, which adds to negative public perception.
This is particularly true in urban areas where lane closures should be minimized. Finding ways to increase the speed at which repairs can be made, and thus decreasing the effects on traffic is critical. One promising solution for reducing construction durations, which the Iowa DOT has been testing and evaluating, is the use of precast concrete pavement and bridge elements.

Over the past decade, several state highway agencies have examined the feasibility and cost-effectiveness of using precast pavement for the accelerated construction or rehabilitation of existing concrete or asphalt pavements. In an initial FHWA-sponsored feasibility study, researchers developed a concept for precast, post-tensioned concrete pavement, which was subsequently tested through a series of demonstration projects. In March 2002, the Texas DOT successfully completed the first such demonstration project on a frontage road near Georgetown, Texas. A second project was completed in California in 2004 (Merritt et. al. 2005), and a third in Missouri in 2005 (Davis et. al. 2006).

A successful precast pavement rehabilitation technique provides many benefits including: 1) expedited construction with almost immediate exposure to traffic, 2) potential for overnight or weekend construction, 3) greater control over concrete batching and curing, increased durability (reduced cracking) from post-tensioning, 4) material savings through reduced pavement thickness, and 5) reduced demands on construction lane closures.

This paper documents the first project in Iowa to use precast pavement, with an emphasis on lessons learned from the fabrication and installation of the precast pavement, as well as some of the observed behaviors of the approach slab during an annual thermal cycle. The project team was a diverse group made up of owners, industry, and academia with expertise in both pavements and bridges. The goal of the project was determine the viability of using precast approach slabs, and the effects on the bridge and approach slab when integrally connecting the approach slab to the bridge. A more in-depth study of the performance of the entire system including the effect of integrally attaching the approach slab to the bridge abutment, the behavior of the girders, and a comparison to cast-in-place approach slabs on a similar parallel bridge have been presented elsewhere (Greimann et. al. 2008).

**Bridge and Precast Pavement Description and Instrumentation**

The northbound bridge of two new side-by-side three span bridges, constructed in 2006 on the new Iowa Highway 60 bypass of Sheldon, IA in O’Brien County, was chosen as the demonstration bridge for the precast approach slab concept. Additionally, both the northbound and southbound bridges were also the test bridges for the evaluation of a new integral approach slab to abutment connection. These were the first bridges in Iowa to tie the approach slab to an I-A bridge. Further, this was the first attempt to use precast pavement for an approach slab in Iowa. The northbound bridge utilized an approximate 77 ft long(at centerline) precast approach slab system (Figure 3.1 and Figure 3.2) that abuts the
mainline pavement at an Iowa DOT standard EF joint. The EF joint is an expansion joint filled with flexible foam and contains 18 in. long 1½ in. diameter dowels spaced every 12 in. at pavement mid-depth that accommodates up to 4 in. of movement. The dowels were inserted into sleeves cast into the ends of the precast panels with greased tubes intended to allow unrestrained movement. Details of the precast approach slab panels are shown in Figure 3.3 and Figure 3.4.

Figure 3.1. Plan view of precast approach slab (northbound bridge)

Figure 3.2. Connection detail for the precast approach slab to abutment
The project team instrumented the south approach slabs, abutments and bridge spans of both bridges to determine the performance of the approach slabs, the effects of attaching the slabs to the bridge, and in the possible range of forces to consider when designing connected approach slabs (in both the bridge and slab). A wide variety of sensors were installed on the bridge and the approach pavement to monitor: temperature, bridge abutment movement (translation and rotation), bridge girder strain changes, approach slab strain changes, post-tensioning (PT) strand losses, approach slab joint relative displacement, and bridge abutment pile strain changes. Figure 3.5 shows the instrumentation plan for the precast approach slab and the south end of the northbound bridge. Readings from the various elements were collected every hour for approximately one year (April 2007 to April 2008). This paper will present only the results relating to the precast approach slab (i.e., the northbound bridge).
Design

The precast post-tensioned approach pavement panels were designed by the Iowa DOT Bridge Office and The Transtec Group. Initially the panels were designed as full roadway width trapezoidal panels that matched the 30 degree skew of the bridge, with transverse pre-tensioning and longitudinal post-tensioning. This configuration was found to be a very complicated and impractical as the panels were large and required special formwork to cast the proper roadway crown and odd shape. Due the complexity of the design, the initial bids for fabrication were very high.

Subsequently, the panels were redesigned to be single-lane-width panels. Twelve standard 14’x20’ panels, 12 in. thick, comprised the majority of project (Figure 3.1). Special trapezoidal panels placed adjacent to the bridge were designed to accommodate the 30 degree skew of the bridge and allow the use of the standard rectangular panels. The use of single-lane panels had other benefits, such allowing two side-by-side panels of uniform thickness to form the roadway crown, while also permitting lane-by-lane construction should only one lane be reconstructed at a time on future applications. The precast panels were post-tensioned in both directions, with a cast-in-place longitudinal joint creating continuity transversely. Similar to cast-in-place approach slabs, a double mat of steel reinforcement was included in the design to meet flexural strength and crack control design criteria; further this was used to accommodate lifting and handling stresses. The concrete design strength of the panels was 7,000 psi. The skewed panels were placed on a neoprene pad at the abutment and anchored in place using vertical stainless steel dowels drilled into the abutment paving notch as shown in Figure 3.2. The 2 in. diameter sleeves cast into the precast panels to receive the dowels were grouted after the dowels were installed.

The post-tensioning was designed to provide enough flexural strength for the approach slab that it could span (unsupported) a void of up to seven feet from the abutment. Two-way post-tensioning also served to tie the panels together longitudinally and transversely. The PT tendons were grouted monostrand 0.6 in. diameter 7-wire post-tensioning strands stressed to 75% of ultimate strength (270 ksi) inside 1 in. diameter PT ducts, spaced 2 ft on center. Longitudinal dead-end PT anchors were located in 9 in. by 9 in. pockets cast into the skewed panels at the abutment (Figure 3.6). The pockets were positioned along the skewed edge of the trapezoidal panels, as close to the bridge abutments as possible. Live-end longitudinal PT anchors were located in the vertical face of the last panel at the end of the approach slab. The transverse PT anchors were located along the outer edges of the approach pavements for the standard rectangular panels. In the larger skewed panel (1A) the same 9 in. by 9 in. pockets used for the longitudinal dead-end anchor were used for the transverse PT dead-end anchors.

The transverse joints between precast panels featured continuous shear keys cast into the panels which helped to ensure vertical alignment between panels and load transfer across the joints prior to application of the longitudinal PT force. Special considerations
were needed for the longitudinal joint because of the crown formed between the panels on either side of the joint and the transverse PT strands which ran across the joint. An open keyway was designed for the joint, and was grouted after the transverse PT strands were feed through the panels and the duct was connected across the joint (Figure 3.7). This grouted joint helped to correct any misalignment of the edges of the precast panels on either side of the joint.

![Figure 3.6](image1.jpg) \textbf{Figure 3.6. Longitudinal PT dead-end pockets along the trapezoidal panel skewed edge}

![Figure 3.7](image2.jpg) \textbf{Figure 3.7. Longitudinal joint prior to grouting}

**Fabrication**

Casting of the panels was not a high production process due to the limited number of panels and their custom nature. Because this was essentially a custom designed project, steel forms were not economically viable for forming the transverse and longitudinal joint keyways. Approval from the Iowa DOT was granted to use wooden forms for the keyways.
Over time these forms swelled and the concrete pushed them out of the vertical plane, resulting in joints with non-vertical faces, as shown in Figure 3.8. This required grinding the vertical faces of several panel edges to ensure the panels would fit together properly (Figure 3.9). Despite the grinding of the face of the keyway, some damage still occurred to the panels during post-tensioning (Figure 3.10).

Match casting is one solution to avoid poorly fitting joints and the cost of steel forms. The negative aspects of match casting include: 1) the space required at the precast plant for a match-casting operation, and 2) the necessity of installing each panel in a specific location rather than as a “standard” panel. Alternatively, adoption and implementation of a standard detail for precast panels and increased use of precast pavement for approach slab construction/repair should make the use of steel forms more economical. The use of steel forms would likely also reduce the time it takes to cast a panel as well as eliminate extra labor to re-finish the panels.

Other issues encountered during fabrication revolved around the PT hardware, which the precaster was not familiar with using. Specifically, the plastic PT duct tended to retain its coiled shape, requiring additional means to keep the duct straight in the forms. This was resolved by inserting #4 reinforcing bar into the ducts to hold them in place until the concrete hardened. Forming the PT pockets also added to the formwork setup time. Two types of pockets were formed in the panels – pockets along the skewed edge to allow access to the dead-end anchors (Figure 3.6) and rectangular pockets to allow access to the PT ducts for instrumentation installation. In addition to forming the pockets, the smaller size of the pockets used in this project led to congestion of reinforcing steel (Figure 3.6). Use of a larger pocket could alleviate the congestion making fabrication simpler and faster.

Figure 3.8. Poorly fitting joint due to inconsistency and displacement of the wooden forms
Prior to setting the panels in place, the base course material, a ¾ in. crushed stone aggregate, had to be graded to the proper elevation and slope. The material was graded by hand to the proper elevation and cross slope and compacted using a portable plate vibratory compactor. After the base course was graded to the proper elevation, a single layer of polyethylene sheeting was placed over the base as a friction reducing membrane. Figure 3.11 shows a photo of the prepared base for the south approach of the northbound bridge.

The panels were set in place by a crane utilizing four lifting anchors cast into the panels. Installation of the panels began with the panels closest to the bridge and continued away from the abutment, with one lane installed at a time (Figure 3.12). Position and elevation were checked at the four corners of each panel using hand measurements and a 2-D laser.
level. Any high or low spots were corrected by grading the base course by hand (Figure 3.13) which for some panels became a somewhat time consuming process.

After each panel was properly placed and dry fitted, it was moved back from the previous panel to allow epoxy to be applied to the joint faces. A two-part gel paste construction epoxy was applied by hand to bond the panels together and provide a full contact surface across the joint. Foam rubber gaskets were installed around each PT ducts to help seal the ducts from grout leakage. The panel was then installed in its final position and lightly stressed to close the joint. Figure 3.14 shows the epoxy being applied to the joint, and the final joint after being pulled closed. After all panels were in placed in one lane they were fully post-tensioned in the longitudinal direction. The process was then repeated for the adjacent lane.

Figure 3.11. Prepared approach ready for panel installation
Figure 3.12. Installation of skewed panel at abutment

Figure 3.13. Base course adjustments
The longitudinal joint between the lanes of panels was filled with a small aggregate concrete/mortar prior to transverse post-tensioning (Figure 3.15). After the concrete was placed and allowed to cure for approximately an hour, the transverse PT strands were tensioned to about 20% of the required tensioning to prevent shrinkage cracks from forming in the joint concrete during curing. After 24 hours, the transverse strands were fully tensioned, completing all post tensioning of the panels. After the panels were post-tensioned in both directions, the PT ducts were grouted to both protect the strands and to increase the transfer of force from the strands to the concrete. A flowable fill (a high slump, one sack cement mix containing only fine aggregates) was used to fill any voids beneath the panels. This under-slab grouting was the final installation step. After all the approach slabs were in place they were diamond ground for evenness and a smooth ride.
One of the primary issues encountered during installation was the many adjustments to the base course required to install the panels at the correct elevation and cross slope. The ¾ in. base course material was difficult to "fine tune" to the proper grade. Based on the experience at the south end of the bridge, the panel installation process was slightly modified for the north end of the bridge. The contractor used steel plates, set at the proper elevation, to support the corners of the panel. While this made the panel installation much quicker, it left a ½ to ¾ in. deep void beneath the slab which required a significantly higher volume of grout for the under-slab grouting operation.

Another issue, which was related to the PT ducts was that the rubber gaskets used to seal the longitudinal PT ducts at the transverse joints were too soft and did not provide a tight enough seal. As a result, grout leaked from transverse joints (Figure 3.16) filling the void beneath the pavement which had not yet been filled with the under-slab grout. This was not an issue with the transverse PT ducts, however, as the joint between the two halves of the duct was encased in concrete. To mitigate this issue on the north end of the project, the under-slab grouting was complete prior to tendon grouting so that there was no place for the PT grout to go. Future applications should consider the use of larger gaskets or a positive coupler connection between the ducts.

![Figure 3.16. Post tensioning duct gaskets and grout leaking beneath the slab](image)

Approach Slab Behavior

The performance and behavior of the precast approach slab was of great interest to the Iowa DOT because it was the first use of precast pavement in Iowa, as well as the first use of an integral approach slab to bridge connection. One of the big questions both the Iowa DOT and researchers had was how the joints between the approach slab and bridge and between the individual precast panels would perform during annual thermal cycles, due to the push/pull of the bridge thermal expansion and contraction. The steel dowels connecting the approach slab to the abutment were expected to keep the joint closed, while the post-tensioning would keep the joints in the approach slab held tightly closed.
To monitor joint movement, crackmeters were installed across five joints: abutment to approach slab, Panel 1 to 2, Panel 2 to 3, Panel 3 to 4, and the expansion joint between panel 4 and the mainline pavement (refer to Figure 3.1 and Figure 3.5). The collected data, shown in Figure 3.17, show that the joints were performing as expected and designed. Negligible relative movement was observed between the panels and between the bridge abutment and panels, while the relative movement at the expansion joint was approximately 1 in. Several important behavior metrics are summarized in Table 3.1.

**Table 3.1. Approach Slab Results**

<table>
<thead>
<tr>
<th>Action</th>
<th>Northbound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slab Load Strain</td>
<td></td>
</tr>
<tr>
<td>Range</td>
<td>120 µε</td>
</tr>
<tr>
<td>Slab Forces</td>
<td></td>
</tr>
<tr>
<td>Range of Force/Stress</td>
<td>2300 kips / 580 psi</td>
</tr>
<tr>
<td>Friction Range</td>
<td>-4 to 4 kips/ft</td>
</tr>
<tr>
<td>Joint Movement</td>
<td></td>
</tr>
<tr>
<td>Abutment Joint</td>
<td>&lt; 0.03 in</td>
</tr>
<tr>
<td>Expansion Joint</td>
<td>0.9 in</td>
</tr>
</tbody>
</table>

Figure 3.17. Precast approach slab joint movements
As the approach slab expands and is pushed by the bridge during thermal expansion, the movement is expected to be resisted by the friction between the approach slab and the base course. The expected result is that the longer the approach slab the larger the friction force would be. Polyethylene sheets were installed under all the approach slabs in an effort to reduce the friction between the slab and the base course. Strain gauges were embedded at the middle of each precast panel with the intention of observing the forces in the slab and for estimating the frictional components. Figure 3.18 shows the total force in the precast slab with respect to location along the length of the slab for a typical warm and cold day. As can be seen there is a slope to the force graph as indicated by the least squares straight line shown in Figure 3.18. Figure 3.19 shows friction force over time. The maximum friction ranged from 8 to -8 kip/ft (coefficient of friction equal to the friction force/slab weight of 1.9) during the monitoring period, however, the majority of the friction ranged from 4 to -4 kip/ft (coefficient of friction of 0.95), which corresponds to a friction angle of approximately $43^\circ$. This is slightly higher than the upper limit presented by Das (1998) of $38^\circ$ for friction between sand and concrete. This suggests that the polyethylene sheeting did not significantly reduce the friction between the approach slab and base course. It may be that, despite the sheeting under the precast slab, the grout formed a rough or uneven surface, due to the voids in the base course.

The force at the expansion joint and abutment was calculated using the best-fit procedure. The precast slab at the expansion joint had slightly larger compression forces than at the abutment, except during the spring and latter part of winter. If the expansion joint were functioning as intended, the measured force at the expansion joint would be close to zero, which is not what was measured (Figure 3.18). Instead there is a substantial change in the slab force during the year (e.g. the difference between a cold day in January and a warm day in September shown in Figure 3.18). The total change in axial stress in the precast approach slab had a range of 580 psi (Table 3.1) which is approximately 8% of the 7000 psi design compressive strength.

The longitudinal PT tendons in the precast slab were also observed as shown in Figure 3.20. A loss of 1.1 kips of post-tension force occurred during the study which is equal to about 2.5% of the initial post-tensioning force. The transverse PT tendons similarly had a loss of approximately 1 kip of post-tension force. It should be noted that the observation period began seven months after the installation and post-tensioning of the precast panels. Thus these loses represent on a small fraction of the total loses.
Figure 3.18. Warm and cold load force with respect to location from the expansion joint in the precast approach slab

Figure 3.19. Slab friction over time
Lessons Learned/Concluding Remarks

The Iowa Highway 60 approach slab project was successful in not only demonstrating the viability of precast approach slabs for use on integral abutment bridges, but also provided information on a number of improvements that can be made for future applications. Key lessons learned are as follows:

1. In terms of fabrication, for a small application such as this, the use of wood forms may be the most cost-effective option, but close attention should be paid to the condition of the forms throughout fabrication to ensure the forms are still within tolerance. If the forms are not properly maintained, the extra labor cost to make the repairs, and the possible onsite delays due to fitting problems could easily negate any cost benefit of the wood forms. Metal forms, produced to the strict tolerances required for this type of application, should be used when possible, especially on larger scale projects to ensure a quality product.

2. The relatively small post tensioning pockets used on this project led to issues during fabrication and installation. The congestion of reinforcing steel around the pockets made installing the steel and block-outs more difficult and time consuming, and required additional attention to ensure good distribution and consolidation of the concrete around the pockets. Increasing the pocket size would spread the steel out, as well as make working with the PT tendons much easier.
3. With regard to panel installation, preparation of the base course was difficult due to the use of larger, ¾ in. aggregate. One solution may be to use a thin layer of a granular/sand material which will allow easier and more precise adjustment of the panels. While using steel plates in the corners of the panel, as was done at the north end of the bridge, made achieving the proper elevation much easier, substantially more grout was used and measures may be needed to ensure that grout is not lost from beneath the slab. The method implemented would need to be project specific depending on which is more valuable – time or material.

4. The gaskets used to seal the PT ducts across the panel joints for this project were found to be inadequate. When the PT tendons were grouted, the grout leaked from the gaskets and filled the voids under the panels. For future applications, larger gaskets or positive couplers that can create enough of a seal to resist the grout pressures would be desirable.

5. The use of half-width (single-lane) panels worked well on this project. Shipping and handling was easier with smaller panels, and fitting the crown of the road was also simpler with the single-lane panels. Single-lane panels would also permit construction to be staged such that only one lane would need to be closed to traffic at a time. The only negatives to using single lane panels are the increased number of panels required and the need for a cast-in-place longitudinal joint. The decision of panel sizes will be project specific. Single-lane panels are better suited for lane-by-lane pavement replacement, while full-width panels may be beneficial on new construction or sites where the site can be completely closed to traffic.

6. The approach slab force was expected to decrease linearly from the joint at the abutment to the EF expansion joint at the mainline pavement as the bridge movements were resisted by friction under the slab. At the EF joint the slab force was expected to be near zero. Instead, the strain was found to be mostly constant along the entire length of the slab, with a notable force present at the end of the slab. It appears that the EF joint is not functioning as designed most likely due to misalignment of the dowels during installation. Since the EF joint is not functioning as designed, the slab cannot freely expand and contract and, as a result, a stress built up in the slab. No evidence was found to indicate that the stress at the EF joint is large enough to cause significant problems, though in the future specific attention should be paid to ensure the joint is installed correctly and functions as designed.
References


CHAPTER 4 GENERAL SUMMARY/CONCLUSIONS

The Iowa Highway 60 demonstration project was successful by demonstrating the viability of the integral approach slab to bridge connection and precast approach slab, and by providing information on improvements that can be made in future projects. Even though it was not determined if connecting the approach slab to the bridge or precast approach slab eliminated the bump, the connection and precast slab performed well without any major issues. The issues encountered on this project were primarily construction related, and can be overcome through awareness of the construction issues and the repeated implementation of integral approach slab to bridge connections and precast approach slabs.

Design

The integral approach slab to bridge connection used on these bridges seems to be a good design, as it held the joint closed the entire thermal cycle. There was no evidence to suggest the connection was not adequate structurally. The current detail is also a good design for repair work as installing a vertical dowel would be the easiest and quickest. Unless a reason, such as future structurally deficiency or non-elimination of the bump, is found the current connection detail should be used.

The post tensioning pockets in the precast panels should be redesigned in future applications. Reinforcing steel was congested around the small pockets making placement of the steel and proper consolidation of the concrete more difficult. Installation of the PT strands was also difficult. The use of longer pockets would make PT strand installation easier and may reduce reinforcing steel congestion.

Fabrication

The biggest issue encountered during fabrication was the tolerances of the wooden forms were not as good as metal forms leading to keyways that did not match as well as designed and intended. Metal forms would be ideal, though if cost considerations require the use of wooden forms, extra attention should be paid to the forms during fabrication to ensure a quality product within specified tolerances.

Installation

Placing the precast panels in the correct location was a time consuming task because the ¾ in. aggregate was difficult to work with and adjust to the proper elevation and cross slope. Two recommended solutions were: (1) using a finer granular/sand material for more precise adjustments, and (2) setting the corners of the panels on steel plates at the correct elevation and then grouting the remaining void under the slab.
Monitoring

No visible distress was noted during the year long structural health monitoring. The trends observed followed annual cycles with shorter term cyclic patterns caused by short term thermal changes and friction ratcheting. A small resisting force was found in the EF expansion joint between the mainline pavement and precast approach slab. The force observed was not significant enough to cause any significant problems. Since movement and expansion of the approach slab is resisted by friction between the base course and the EF joint it is recommended that a monitoring system be installed on future projects where the integrally connected approach slab length exceeds 80 ft. This project was limited to results on approach slabs less than 77 ft long.