# Performance Evaluation of Recent Improvements of Bridge Abutments and Approach Backfill

*Sponsored by:*
The Iowa Highway Research Board
TR-736

**Final Report**
November 19, 2018

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## STATEMENT OF NON-DISCRIMINATION

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<tr><td colspan="3">16. Abstract<br>A bump at the approach of a bridge is an undesirable characteristic that causes bad ride quality and high maintenance costs. To mitigate bump formation, the Iowa Department of Transportation (DOT) revised its standards according to a former study completed in 2005. This current study evaluated the effectiveness of these revisions by investigating eight bridges in Iowa constructed after 2005, two of which were stub bridges and six of which were integral bridges. One stub bridge and three integral bridges were subjected to a detailed investigation including visual and borescope inspections, coring, GPR assessment, and elevation surveying. The other four bridges were subjected to visual and borescope inspections only. The data showed that components of the abutments were generally in good condition and the joints in the approach slabs were in the poorest condition; many joints were in need of maintenance. GPR assessment detected voids under the approach slabs leading to the integral bridges, and showed that voids were more extensive at the shoulders than at the centerline of the pavement. The stub bridge investigated with GPR had relatively limited voids. Borescope inspection of the access ports confirmed the presence of the voids in the integral bridges. The relatively large extent of voiding under the integral bridges is attributed to cyclic compression of the backfill due to abutment movement and failure of the joint between the barrier and the approach slab due to large differential movements between the two components, which would permit bridge runoff to enter and erode the backfill. Despite the presence of the voids and failed joints, the bridge approaches performed well. Several recommendation were made to improve inspection practices and performance of approach slabs, including the use of GPR assessment to investigate voiding beneath the approach slabs, a new design detail for stub abutments, and sealing or eliminating the gap between the barriers and approach slabs.</td></tr>
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# EXECUTIVE SUMMARY

Approach slabs are designed to transfer traffic from the roadway to the deck of a bridge smoothly. However, a bump at either end of the bridge deck often forms due to differential settlement between the approach slab and the bridge. To mitigate bumps at bridges, the Iowa Department of Transportation (DOT) revised their standard plans based on results from a previous study sponsored between 2002 and 2005, summarized in a report by White et al. 2005. The current study evaluates the effectiveness of those revisions. It includes a literature review of other states' best practices, review of Iowa DOT's new design and construction documents for eight bridges constructed after the 2005 study, and visual inspection and nondestructive evaluation of the eight bridges and their approach slabs. Nondestructive techniques used include borescope inspection, surveying, and ground penetrating radar (GPR) assessment. The data was used to identify issues unaddressed by the current design and provide recommendations for potential improvements.

The eight Iowa bridges that were inspected were sampled to investigate both stub and integral bridges, and bridges abutting hot-mixed asphalt (HMA) pavements and Portland cement concrete (PCC) pavements. All bridges were built between 2006 and 2015, after the 2005 study. A detailed investigation was performed on four of the bridges. The detailed investigations consisted of:

- A visual inspection wherein the bridge components were given condition ratings ranging from poor (in need of maintenance) to good (negligible distress present);
- Coring in order to confirm the presence and measure the depth of the voids;
- Borescope investigation wherein the presence of voids was confirmed by snaking a camera through the access ports and cores;
- GPR assessment to determine the extent of voids beneath the approach panels; and
- Surveying to measure the elevation profiles of the approach slabs and calculate the bridge approach index (BI) of the bridges.

The remaining four bridges were subjected to visual inspection and borescope inspection of the access ports only. Components that were inspected visually included the approach pavement surfaces, expansion joint at the bridge deck and joints between approach slab panels, barriers at the shoulders, abutment wings, berm slopes, and subdrain outlets.

Visual assessment results showed that the abutment wings, rip-rap slopes, and subdrain outlets tended to be in good condition and the pavement surfaces of the approach slabs were in adequate to good condition. The joints and barriers were in the poorest condition and many had vegetation, failed sealant, spalling, and/or raveling. The majority of the expansion joints and joints between the roadway and approach slab differed in width from the design standards, and differences varied from -2.25 inches to 3 inches.

The GPR and borescope investigations showed that the integral bridges tended to have extensive voids adjacent to the bridge abutments. According to the GPR data, voids extended further under the approach slab at the pavement shoulder adjacent to the barriers than at the centerlines. This is likely caused by cyclic compression and settlement of the backfill material as the abutment moves. In addition, failure of the joint between the approach slab and the barrier increases the problem as it allows for backfill erosion. These two elements are not rigidly attached and can undergo differential movements in both the longitudinal and vertical directions. The abutment and wingwalls are integral with the bridge superstructure and, therefore, are subject to movement as the bridge expands and contracts under thermal effects. The barriers are rigidly connected to the wingwall on each side and, therefore, also move with the bridge. The approach slabs are mainly supported on grade and the paving notch at the back of the abutment. As a result, fracture or extreme sealant failure had occurred at many of the joints between the approach slab and the barriers, and the gaps

were large enough to permit bridge runoff to enter the backfill underneath the approach slab and erode the material. In comparison, the stub bridges experienced very little voiding. The results from Bridge 5111.5O034 (stub bridge built in 2006) shows that voids under the approach were limited when compared to the voids detected at the integral abutment bridges, indicating that stub abutments are less susceptible to backfill erosion and settlement.

Despite the presence of large voids, all of the bridge approaches were performing relatively well and BI values calculated from the elevation surveying data were relatively small. The maximum settlement was estimated as 1.0 inch and the maximum differential settlement measured between the bridge and approach slab at the bridge joint was 0.4 inch.

These results led to the following recommendations:

- A significant number of the joints were in poor condition and required maintenance, indicating that a more frequent inspection and/or maintenance schedule may be required.
- While the access ports were helpful with regards to borescope inspection and visual confirmation of voids, they were commonly partially or fully blocked by soil/backfill or other debris. To improve the reliability of borescope inspections, more stringent procedures for sealing the access ports after original construction and after each use and a method to clean the access ports should be developed.
- GPR inspection and proper interpretation of the data was an effective method for identifying void conditions beneath the approach slabs. It is recommended that future inspections of void conditions rely on GPR surveys as the primary data collection method.
- Stub abutment bridges have less voids under the approach slabs compared to integral abutment bridges. Future study is recommended to confirm this behavior and determine if stub abutment designs should be preferred to integral abutment designs. A new design detail for a ‘modified stub abutment’ is also proposed.
- The gaps between barriers and approach slabs should be addressed in order to prevent erosion under integral bridges. Two proposed options are: a) for existing bridges, the gap should be sealed using a material that can tolerate the movement of the two components and the runoff should be redirected away from the joint, and, b) new bridges should be designed such that the barriers are rigidly connected to the approach slab but not the wingwalls.

## ACKNOWLEDGEMENTS

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# CHAPTER 1. INTRODUCTION

## 1.1 Background

Approach slabs are used to transfer traffic from the roadway to the bridge smoothly. However, due to construction and design flaws, differential settlement or bumps are often present at the end of bridges. The presence of a drop at the end of the bridge (bump) has been an issue with bridges across the United States for decades. This bump causes several problems, including a reduction in the ride quality (i.e., distraction of drivers, reduction in steering response), bridge and joint damage, increase in maintenance costs, and adverse effects on the image of transportation agencies (Briaud et al. 1997). Differential settlement at the bridge joint commonly occurs due to the presence of voids beneath the approach slab. Voids can be caused by the longitudinal displacement of an integral abutment, which crushes the backfill material behind it. They may also be caused by poor compaction of the original backfill material. Regardless, they are exacerbated by erosion and permit the approach slab to settle below the level of the bridge deck.

In order to reduce or eliminate this problem, several DOTs conducted research studies and/or updated their approach slab design and construction procedures to address the bump at the end of the bridge and improve the performance of approach slabs. Main improvements include the specification of backfill materials and drainage systems to limit settlement and erosion under the slabs. In addition, approach slabs are now constructed as reinforced concrete sections that are capable of spanning potential voids developed underneath the slab without significant damage due to poor structural capacity.

Iowa Department of Transportation (Iowa DOT) sponsored a previous research study between 2002 and 2005 to identify best practices for design, construction and repair of bridge approaches. The study found that most of the new Iowa bridges they observed did not meet the Iowa DOT specifications, and that the specifications themselves were insufficient. The granular backfill was not compacted well enough and had an unsuitable moisture content. Expansion joints were not sealed tightly enough to prevent water intrusion. Subdrains were not always constructed as designed and often became blocked or collapsed while surface drains were partially to fully blocked by debris. These deficiencies caused voids to develop within a year of construction, which led to faults in the approach slabs, slope protection failure, and exposure and corrosion of the piles. While Iowa DOT did not incorporate every recommendation by White et al. (2005), design details were revised and the following standards are currently in effect:

- An expansion joint between 2 and 3.5 inches, depending on the length of the bridge, is to be between the approach slab and the bridge deck.
- Granular backfill should be used behind the abutment. Elastic tire buffings are to be used under the joint sealant to prevent void initiation.
- A polymer grid or two layers of subgrade paper should lay between the approach slab and the paving notch to prevent bonding.
- A 4-inch perforated subdrain surrounded by porous backfill should be incorporated at the roadway end of the approach slab.

Other state DOTs have struggled with bump formation as well and some of their best practices coincide with the standards set by Iowa DOT, such as the use of porous and/or granular backfill for better drainage and erosion resistance. Notable deviations include using a paving seat instead of a paving notch to improve constructability, incorporating the expansion joint between the roadway and the approach slab instead of the joint at the bridge deck and approach slab, and wrapping a geotextile around the subdrain and its porous media.

This study aims to evaluate the performance of the design and construction practices that have been implemented to reduce the bump at bridge abutments and approach slabs. The study includes review of the design and construction documents for selected bridges from the 2005 field trials; selection of specific bridges for field evaluation; inspection and nondestructive evaluation of approach slabs' settlement; and field inspection of bridge approach surfaces and components. The results of this study are presented in this final report, which provides documentation of the field evaluation efforts with supporting observations, photographs and measurements.

## 1.2 Scope of Work

The main objective of this study is to evaluate the performance of modifications that have been made in the design and construction of approach slabs in Iowa since 2005. The study includes review of the design and construction documents for selected bridges from the 2005 field trials and selection of specific bridges for field evaluation. Settlement of the approach slabs selected for evaluation was investigated using nondestructive evaluation techniques including borescope (cable snake camera) inspection, surveying, and GPR assessment. Visual condition assessment of the approach slab, joint, and the end of the bridge structure for cracking and related distress was completed, including evaluation of the approach pavement, roadway shoulder, abutment wings, abutment footing face under the bridge, subdrain outlets for the abutment backfill drainage, berm slope erosion conditions, and joint type and condition. The results of the field inspections were used to assess the performance of the bridge approaches and identify where issues with the current design may exist. Recommendations for potential improvements in the bridge approach design and construction are presented.

## 1.3 Layout of Report

This report includes four chapters, including Chapter 1 - Introduction.

Chapter 2 provides background and review of available literature on mitigating bumps between approach slabs and bridges. The chapter summarizes the findings of the study conducted by Iowa DOT in 2005 for reference and includes an overview of the standards and practices implemented in several other state DOTs to prevent bumps. The information is categorized by general approach slab design, with particular focus on the connection between the approach slab and the abutment, drainage details, and expansion joint details.

Chapter 3 describes the methods used to evaluate the bridges in the field and provides the results of the inspections. Methods include visual inspection, borescope, ground penetrating radar, and elevation surveys. The results are categorized by bridge and then by method. A brief description of the bridge is given, followed by the observations from the visual inspection, then the results from the borescope, core(s), and GPR, and finally the elevation maps resulting from the elevation surveys. A discussion of all the collected results is also included.

Chapter 4 summarizes the findings, conclusions, and recommendations for future implementation of design revisions for bridge approaches of integral bridges.

# CHAPTER 2. LITERATURE REVIEW

## 2.1 Background

Approach slabs are meant to smoothly transfer vehicles from roadways to bridges. Depression and cracking of approach slabs is a historic problem due to settlement of construction backfill placed against bridge abutments or due to differential settlement. The approach slabs often settle below the level of the bridge deck which creates a bump. These bumps hinder ride quality, cause damage to the vehicles and bridge decks, and are costly to repair and maintain. Many different approach slab designs have been implemented to reduce this problem. In modern designs, one end of a structurally designed steel reinforced approach slab rests on the abutment and the other end rests on a sleeper slab or original subbase. The majority of the reinforced approach slab typically rests on the backfill behind the abutment, at least initially.

Two different types of abutments are generally used by state DOTs, stub abutments and integral abutments. The primary difference between stub and integral abutments is how they connect to the bridge superstructure to address global length changes caused by seasonal temperature variations. Figure 2.1 shows typical schematics for a stub abutment and an integral abutment.

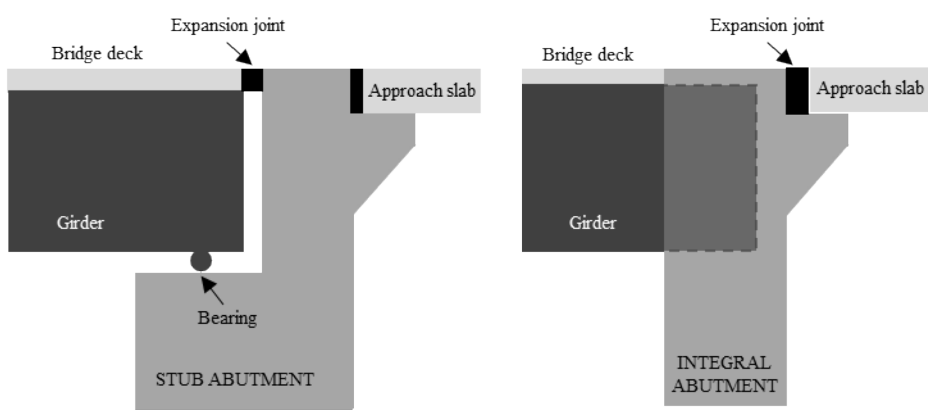



*Figure 2.1. Schematics of a stub abutment (left) and an integral abutment (right).*

Stub abutments, also known as fixed or conventional abutments, use steel bearings to support bridge girders, as represented by the roller under the girder in Figure 2.1 (left). An expansion joint is installed between the bridge deck and the abutment to permit expansion or contraction of the deck without deflecting the abutment. A second joint is installed between the approach slab and the abutment. The approach slab is rigidly connected to the paving notch by steel dowels.

Integral abutments, also known as movable abutments, are configured such that bridge girders are embedded into and composite with the concrete abutment. As such, the abutment must deflect as the girder length changes. The joint installed between the abutment and the approach slab serves as both a connecting joint and an expansion joint. A layer of bearing material is placed between the approach slab and the paving notch to facilitate relative movement. Several state DOTs have specified that a regular construction joint connect the abutment and the approach slab, causing the approach slab to slide with the abutment. The expansion joint is then installed on the other end of the approach slab adjacent to the roadway pavement.

Recently, integral abutments have been preferred to stub abutments. The exposed steel and bearings of stub abutments are more expensive to build and maintenance can be costly and difficult. They are susceptible to deterioration due to exposure to deicing salts. In comparison, integral abutments do not require girder bearings and also provide added protection to the ends of the girders from deicing salts and inhibit corrosion initiation. They can also reduce the number of piles required and provide better seismic performance (White et al. 2005). However, integral abutments have maintenance concerns as well. The primary concern is backfill erosion. Because the abutments move, the backfill experiences cyclic loading and plastic deformation. The deformation initiates a void, which facilitates water movement and encourages erosion and subsequent void growth. The void exacerbates the differential settlement between the abutment and the approach slab, causing larger bumps between the bridge deck and the approach slab. Both abutments are susceptible to differential settlement and bumps due to inaccurate geotechnical analysis and modeling. Some groups have proposed designing shallow foundations for integral abutments instead of deep piles to prevent differential settlement (White et al. 2005).

## 2.2 Standard Practice in Iowa

The Iowa DOT funded a study between 2002 and 2005 to study approach slab performance (White et al. 2005). The objectives of the study were to understand the causes of approach slab bumps in Iowa bridges, develop threshold criteria for their maintenance or repair, identify current practices used to prevent bumps, and recommend improved design, construction, and maintenance practices to the Iowa DOT. This past project was done in five parts: a literature review of other states' practices, field inspection of existing bridges in Iowa, characterization of the backfill materials used and their properties, analysis of the structural failure of the paving notch and approach slab configuration, and characterization of the severity of bumps observed.

In their literature review, White et al. (2005) found that integral abutments (also known as movable abutments) are preferred because they are more cost-effective and less susceptible to deterioration compared to non-integral abutments (also known as stub or fixed abutments) (Horvath 2000; Hoppe and Gomez 1996). Because the bridge girders are embedded in integral concrete abutments, the abutment experiences longitudinal deflection when the girders expand or contract from temperature changes. This deflection causes void development at the abutment-backfill interface and differential settlement between the bridge and the approach slab (Arsoy et al. 1999). The voids make the embankment behind the abutment more susceptible to erosion and settlement, encouraging approach slab settlement and the formation of a bump. Strategies for preventing bumps included (Briaud 1997; Wahls 1990):

- Improving the foundation soil, most commonly by preloading;
- Using backfills that are easily compacted, erosion-resistant, and elastic and have limited amounts of fines;
- Including reinforcement and compressible and collapsible materials in embankments;
- Using footings or other shallow foundations instead of piles for the abutment; and
- Designing good surface and subsurface drainage.

Integral reinforced approach slabs are intended to mitigate bump development by minimizing differential settlement, permitting expansion, and preventing surface water from entering the embankment. Approach slabs in the states varied from 10 to 40 feet in length and 8 to 17 inches in thickness in 1999 (Hoppe 1999). They were often mechanically connected to the abutment with reinforcing steel dowels. The joint design between the approach slab and the bridge deck varied from 0.5 to 2 inches in width. Iowa reported using a 1-inch joint opening filled with expansion material. The Iowa DOT specified that the backfill should have 100% passing the 3-inch sieve, 20 to 100% passing the No. 8 sieve, and 0 to 10% passing the No. 200 sieve and that it was to be compacted to 95% of its maximum dry density. White et al. (2005) stressed that no moisture content was specified. Finally, Iowa used a perforated drain pipe surrounded by porous backfill as a subdrain almost exclusively.

White et al. (2005) investigated 74 completed bridges across all six Iowa districts and eight new bridges under construction. They found that most of the new bridges did not fully meet Iowa DOT's specifications and that the specifications themselves were insufficient. The granular backfill was not compacted at five of the eight new bridges, and there was no porous backfill around the subdrain at seven of the new bridges. Additionally, the backfill material was susceptible to bulking because it was placed at moisture contents between 3% and 5%. Bulking is a phenomenon in which the material's volume increases because of water adhering to sand particles, and the tensile stresses between water and soil particles resist compaction. It typically occurs at moistures contents between 3% and 7%. White et al. (2005) found that the backfills used in Iowa experienced 6% collapse when saturated. Based on their observations of the completed bridges, the insufficient compaction and moisture control caused voids to develop within a year of construction. The voids decreased with distance from the abutment and varied in depth from 12 to 0.5 inches. Erosion caused further void development under approach slabs, which led to faults in the slabs, slope protection failure, and exposure and corrosion of the piles. Filling the voids with grout was a common maintenance practice in four of the districts; however, this practice was found to be ineffective because it did not prevent further settlement or backfill erosion. Some approach slabs had been raised using an injectable liquid polyurethane foam. This maintenance method was economical and effective in the short-term, but the study did not conduct long-term monitoring to verify the success of this method.

Poor water management was a recurring issue as well. Flexible foam and recycled tire joint fillers did not seal the expansion joints correctly, permitting water intrusion. Some of the subdrains in the completed bridges were blocked, dry, or collapsed and subdrains in the new bridges became filled with soil during and after construction. When characterizing the backfill materials, White et al. (2005) found that about 70% of the granular backfill particles were smaller than the size of the perforations in the subdrains. This made the backfill highly erodible and plugged the drains. On the surface, drains parallel to the pavement and covered in a grid tended to experience debris build-up and became partially to fully blocked. White et al. (2005) repeatedly compared these problems to a successful drain inlet that was built into the curb of one bridge's approach. This drain had no grid and was almost perpendicular to the pavement surface, and as a result did not experience any blockage due to debris.

In their studies on backfill properties, White et al. (2005) identified several practices that could improve both the compaction of the backfill and the drainage in the embankment. They found that the granular backfill materials currently in use were not susceptible to bulking when placed at moisture contents greater than 8%, and that limiting the percent passing the No.8 sieve to 60% would prevent erosion. Alternatively, porous backfills can be used that are not susceptible to bulking and are naturally resistant to erosion. The study also compared the drainage performance of the current granular backfill material to porous backfill, a geocomposite drainage system STRIPDRAIN 75, and tire chips using a drainage model built in-house. The granular backfill had the smallest drainage ability with a flow rate of 32 $cm^3/s$. The porous backfill increased the drainage rate almost by a factor of 3, the geocomposite drainage system increased the rate by a factor greater than 10, and the tire chips increased the rate by a factor greater than 17.

Finally, White et al. (2005) observed that paving notches in the new bridges were poorly-constructed. They often had a sloped top surface and poor consolidation. In one bridge experiencing maintenance, the paving notch had broken off and the approach slab was supported on only 0.5 inches of concrete. This supported the results of their analytical investigation of the paving notch and approach slab configuration. Failure by direct vertical shear through the notch, tension tie yielding, concrete strut crushing, and localized bearing failure was evaluated. While the simulations showed that the design was sufficient, they found that shear failure was the most likely mode of failure and recommended increasing the size of vertical steel reinforcement from No. #5 rebar to No. #7 rebar in non-integral bridges. They concluded that premature failure was caused primarily by poor materials or workmanship.

To evaluate the condition of approach slabs, the study classified bumps by severity using three systems: the International Roughness Index (IRI), Iowa DOT ratings based on riding quality, and a new rating system devised by White et al. (2005) that combined the IRI and a Bridge Approach Index (BI) value. The results are summarized below in Table 2.1 and show that the Iowa DOT and the new rating systems are much more stringent than the IRI. According to the new rating system, 92% of the bridges needed maintenance.

*Table 2.1. Distribution of bump ratings for bridges across Iowa according to the three systems used by White et al. (2005).*

| Rating | Condition Evaluation System | | |
|---|---|---|---|
| | IRI | Iowa DOT | New (BI & IRI) |
| Very good | 7.7% | n/a | n/a |
| Good | 57.7% | 3.8% | 4% |
| Fair | 23.1% | 15.4% | 4% |
| Poor | 11.5% | 63.4% | 8% |
| Very poor | 0.0% | 15.4% | 84% |

In summary, White et al. (2005) recommended the following to Iowa DOT for the prevention of bumps in new bridges:

- Use a porous backfill and a geocomposite drainage system behind abutments;
- Use the surface drain inlet included in the curb instead of gridded surface drain inlets parallel to the pavement;
- Change the maximum percent passing the No.8 sieve to less than 60% in backfill materials;
- Specify that backfill materials have a moisture content between 8% and 12% during placement and compaction;
- Connecting the approach slab to the abutment and/or bridge deck and eliminating the expansion joint;
- Incorporate a sleeper beam with a 2-inch construction joint at the far end of the approach slab;
- Replace No. #5 vertical rebar with No. #7 rebar in abutments for non-integral bridges; and
- Add geotextile reinforcing layers to backfill or a thick layer of tire chips behind abutments.

A review of the specifications used by Iowa DOT today indicate that some of the recommendations from White et al. (2005) have been incorporated while others have not (Iowa DOT 2017). The expansion joint remains between the approach slab and the bridge deck, although bridges containing the expansion joint at the sleeper slab have been constructed in Iowa when the abutment is tied to the approach. The width of the joint depends on the length of the bridge and varies between 2 inches and 3.5 inches. Tire buffings are used under the joint sealant above the paving notch for better elasticity. A polymer grid or two layers of subgrade paper are placed between the approach slab and the paving notch to prevent bonding. In non-integral (fixed) abutments, a diagonal rebar is used to tie the approach slab and the paving notch. A granular backfill and subdrain is still used behind the abutment, but a detailed design is not specified. A 4-inch perforated subdrain surrounded by porous backfill is used at the far end of the approach slab.

## 2.3 Practices of Other State DOTs

The practices of nine other states regarding approach slabs and abutments, particularly integral abutments, were reviewed. The states included were Illinois, Washington, Minnesota, Wyoming, Virginia, New York, Massachusetts, California, and New Mexico. Drainage, backfill, and joint details were reviewed for comparison to the Iowa state practice.

The Illinois DOT specifies porous granular backfill. A 6-inch diameter surface drain is used at the boundary between the parapet and the bridge. In the standard details, the metal covering over the inlet has no grid. The paving notch and approach slab are connected using a No. #5 vertical rebar. A formed construction joint is specified between the bridge and the approach slab, and is to be filled with a bridge relief joint sealer. The expansion joint is above the sleeper slab, which must be 1.75 inches in width at 50˚F (IDOT 2017a; IDOT 2017b; IDOT 2016).

The Washington State DOT uses gravel backfill behind abutments and requires a subdrain surrounded by a second gravel backfill material at the corner between the abutment and the footing. Weep holes are included in the abutment walls to drain water from the backfill. All bridge runoff is to be collected at the abutments and carried at least 10 feet beyond the approach slab (WSDOT 2007). The approach slab rests on a seat instead of a paving notch, as shown in Figure 2.1. The seat must provide at least 10 inches of support. Expansion joints have been identified as the component most susceptible to damage and as such the Washington State DOT emphasizes minimizing the number of expansion joints and using semi-integral construction. In L-type (semi-integral) abutments, the seat and the approach slab are connected with a bent No. #5 rebar. Alternatively, semi-integral abutments may be connected by a stop type coupler spanning an expansion joint filled with expanded polystyrene. Expansion joints are classified as small if they are less than 1.75 inches in width, large if they are greater than 5 inches in width, and medium otherwise (WSDOT 2007).

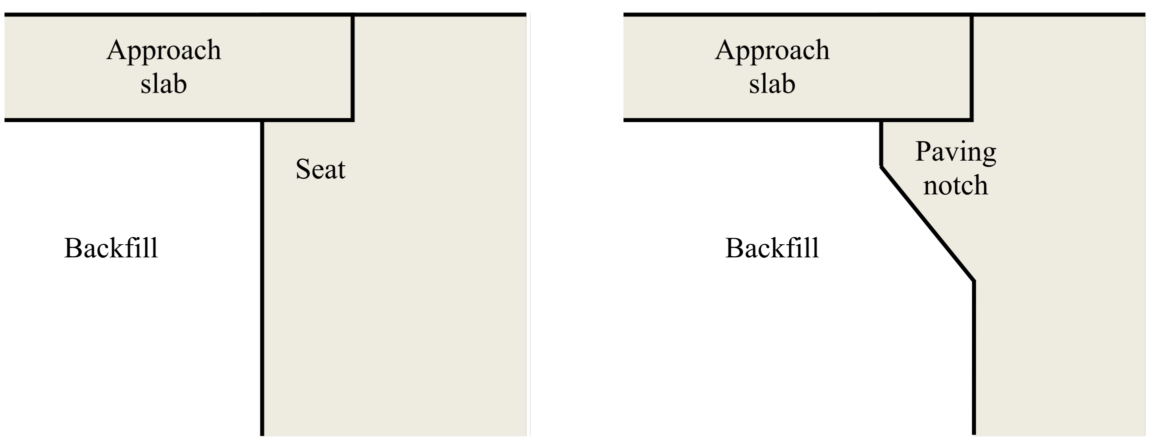



*Figure 2.2. Schematics showing the paving notch versus the seat configuration.*

The Minnesota DOT uses a granular backfill and a membrane waterproofing system behind the abutment. Subdrains are required at the bottom of the fill and around the sleeper slab. Options are provided, but a 4-inch diameter perforated thermoplastic pipe with a geotextile wrap embedded in or placed under fine filter aggregate is typical. A perforated pipe runs transversely across the approach slab just before the abutment to channel surface water. Surface drain inlets are parallel to the road surface and covered by grids (MnDOT 2018). The expansion joint is at the interface between the approach slab and the roadway. A construction joint is used between the approach slab and the bridge deck and a diagonal tie connects the abutment and the approach slab. A 12-mil polyethylene sheet is required under the approach slab so it can slip longitudinally with the abutment (MnDOT 2018; MnDOT 2016).

The Wyoming DOT requires porous backfill and suggests crushed gravel or rock or manufactured sand. The backfill is to be placed and compacted in lifts between 8 inches and 2 feet in thickness and includes geotextiles. A 6-inch diameter perforated pipe wrapped in geotextile is used as a subdrain and placed at the top of the footing and adjacent to the abutment backwall. The reinforced approach slab is typically 25 feet long and is supported by a paving notch and a sleeper slab. The expansion joint is located between the roadway and the approach slab and a 4-mil polyethylene sheet is placed under the approach slab to facilitate longitudinal movement (WYDOT 2015).

The Virginia DOT permits the approach slab to be on grade or buried. Buried slabs are to be used if the bridge deck slab is extended. They lay at a prescribed depth below the finished grade, but still rest on a pavement seat and span the distance from the roadway to the abutment like an approach slab on the surface. If the slab is buried, a 6-inch diameter PVC perforated pipe surrounded by porous stone and wrapped in geotextile is used as a subdrain along the backwall and above the slab. A geocomposite wall drain and a pipe subdrain are used together regardless of whether the slab is buried or not. A layer of expanded polystyrene separates the geocomposite wall drain from the abutment (VDOT 2018).

Full integral abutments are the preferred method of construction in Virginia (VDOT 2018). The approach slab sits on a seat and the flange of a sleeper slab shaped in an upside-down T. The pavement and the approach slab rest on the flanges of the T opposite each other. A galvanized steel diagonal tie connects the approach slab and the abutment. All integral abutments have expanded polystyrene wrapped in geotextile filter fabric behind them at a thickness to accommodate the expansion. However, the expansion joint is located between the approach slab and the T-shaped sleeper slab. The joint has a width 0.25 inches greater than the maximum movement of the integral abutment, and the maximum abutment movement permitted for a full integral bridge is 1.5 inches. Lubricant and bedding pads are placed between the approach slab and the sleeper slab (VDOT 2012).

The New York DOT considers integral abutments to be the first choice and semi-integral abutments the second-best choice (NYSDOT 2017). A waterstop and/or drain is required at the backwall of the abutment and they emphasize good surface drainage. Approach slab lengths are generally equal to the height of the abutment over the cosine of the skew angle, with a minimum length of 10 feet and a maximum of 25 feet permitted. Similar to Washington and Virginia, New York requires that the approach slab sit on a seat rather than a paving notch and be connected to the bridge deck by a cold construction joint with a silicone sealant. No. #5 diagonal bars connects the approach slab and the abutment. The approach slab is expected to translate and rotate and so the New York DOT advises to avoid rigid connections with the superstructure (NYSDOT 2017). Only bridges greater than 100 feet in length require an expansion joint and the expansion joint is located over the sleeper slab. The approach slab is separated from the sleeper slab by a compressible foam joint sealer and separated from the subbase by a polyethylene curing cover (NYSDOT 2017; NYSDOT 2008a; NYSDOT 2008b; NYSDOT 2010a; NYSDOT 2010b).

The Massachusetts DOT specifies three different types of approach slabs. All are to have controlled-density, non-excavatable fill beneath them and water from the fill is channeled out of the abutment by PVC weep holes. A Type I slab is connected to the abutment by a No. #6 vertical dowel embedded in a 2-inch diameter PVC sleeve that runs through the approach slab and is filled with non-shrink grout. A water-proof membrane is used in conjunction with this configuration. A Type II slab is connected to the bridge deck by horizontal No. #4 rebar. Two layers of tar paper separate the paving notch from the approach slab. Finally, a Type III approach slab is a combination of the Type I and Type II slabs in that it uses vertical No. #6 rebar and horizontal No. #4 rebar to connect to the abutment. The primary difference is that the vertical rebar is embedded in concrete instead of grout (MassDOT 2013).

The California DOT (Caltrans) specifies that a 3-inch diameter slotted plastic pipe in a treated permeable base and lined by geocomposite drain and filter fabric be placed in any corner that water reaches. Geocomposite drains are located under the interface between the barrier and the slab and at the paving notch as well. The approach slab is connected to a seat by a vertical No. #5 rebar if the abutment is non-integral. An anchor assembly that runs through an expansion joint is used to connect the approach slab and the abutment if the abutment is integral. Expanded polystyrene is used as a filler in the expansion joint (Caltrans 2017).

The New Mexico DOT conducted a study on bridge approach slabs in 2006 consisting of a literature review and field evaluation of existing bridges (Lenke 2006). They concluded that bumps were primarily caused by insufficient investigation of foundation behavior and construction practices, which caused unexpected embankment settlement. To combat this, they recommended using a higher-quality backfill with less than 20% passing the No. #200 sieve in areas within 100 feet of the abutment. The backfill was to be compacted to 95% of the modified proctor density and placed in thinner lifts. Because paving notches prevented compaction behind abutments, they suggested using alternatives such as seats. They also recommended increasing the frequency of QA/QC testing of the backfill and embankment. In the field evaluation, Lenke (2006) observed that mechanically stabilized earth (MSE) walls helped prevent approach slab settlement. This was attributed to the better lateral constraint, tie-back straps, and use of free-draining material, which was easier to compact. Currently, New Mexico requires that the backfill conform to AASHTO A-1-a and specifies the order in which the fills are to be placed so that compaction is not compromised. According to Lenke (2006), the literature review showed that as the length of the approach slab decreases, the magnitude of the bump increases. Former studies suggested a maximum slope of 1/200 for the approach slabs, a minimum length of 20 feet for approach slabs, and a minimum length of 5 feet for a sleeper slab. Lenke (2006) endorsed these recommendations as well. Drainage is primarily handled using concrete channels and inlets on the surface. Lenke (2006) recommended that drainage be on the shoulder and as far away from the approach slab and roadway lanes as possible. Better maintenance of the approach slab joints and drains was required since water intrusion and concrete alkali-silica reaction (ASR) were both potential issues in the area. To further prevent water infiltration, Lenke (2006) suggested tying the approach slab into the wingwalls and barrier walls. However, the New Mexico bridges did not have major issues with erosion, indicating that the drainage systems in place worked well.

The practices described above may be categorized according to general approach slab details, drainage details, and expansion joint details. Tables 1 through 3 below summarize the information, respectively.

*Table 2.2. Details regarding approach slab structure according to the state DOT specifications. -/- means no information was found. *Based on suggestions from Lenke (2006).*

| State | Approach slab length | Notch or seat | Tie between approach slab and abutment | Material between approach slab and sleeper slab/subbase | Details behind abutment | Backfill material |
|---|---|---|---|---|---|---|
| IA | -/- | Notch | Diagonal in fixed abutments | -/- | -/- | Granular |
| IL | -/- | Notch | Vertical No. #5 | -/- | -/- | Porous granular |
| WA | -/- | Seat | Bent No. #5 in semi-integral abutments | -/- | -/- | Granular |
| MN | -/- | Seat | Diagonal | 12-mil polyethylene sheet | Membrane waterproofing system | Granular |
| WY | 25 ft | Notch | -/- | 4-mil polyethylene sheet | -/- | Porous crushed gravel/rock or manufactured sand |
| VA | -/- | Seat | Diagonal, galvanized | -/- | Expanded polystyrene wrapped in geotextile | -/- |
| NY | 10 to 25 ft | Seat | Diagonal No. #5 | Polyethylene curing cover | -/- | -/- |
| MA | -/- | Notch | Vertical No. #6 and/or horizontal No. #4 | -/- | -/- | Controlled-density, non-excavatable fill |
| CA | -/- | Seat | Vertical No. #5 if fixed abutment; anchor assembly if integral abutment | -/- | -/- | -/- |
| NM* | > 20 ft | Seat | -/- | -/- | -/- | AASHTO A-1-a, compacted to 95% of modified proctor density |

*Table 2.3. Drainage details according to the state DOT specifications. -/- means no information was found. *Based on suggestions from Lenke (2006).*

| State | Subdrain description | Subdrain location(s) | Surface drain description |
|---|---|---|---|
| IA | 4-in. diameter, perforated pipe in porous backfill | Behind abutment | -/- |
| IL | -/- | -/- | 6-in. diameter inlet parallel to road surface |
| WA | Surrounded by gravel backfill | Corner of abutment and footing | Runoff carried at least 10 ft beyond approach slab |
| MN | 4-in. diameter, perforated, thermoplastic pipe wrapped in geotextile and embedded in/under fine filter aggregate | Bottom of fill and around sleeper slab | Parallel to road |
| WY | 6-in. diameter, perforated pipe wrapped in geotextile | Behind abutment | -/- |
| VA | Geocomposite wall drain; 6-in. diameter PVC perforated pipe in porous stone, wrapped in geotextile | -/- | -/- |
| NY | -/- | -/- | Waterstop; drains at backwall |
| MA | -/- | -/- | -/- |
| CA | 3-in. diameter, slotted plastic pipe in treated permeable base and lined by geocomposite filter fabric; geocomposite drains | Paving notch and under interface between barrier and slab | -/- |
| NM* | -/- | -/- | On shoulder, as far away from slabs as possible |

*Table 2.4. Expansion joint details according to the state DOT specifications. -/- means no information was found.*

| State | Expansion joint location | Expansion joint width |
|---|---|---|
| IA | Between approach slab and bridge deck | 2 to 3.5 inches |
| IL | Between approach slab and roadway | 1.75 inches at 50˚F |
| WA | -/- | < 1.75 inches to > 5 inches |
| MN | Between approach slab and roadway | -/- |
| WY | Between approach slab and roadway | -/- |
| VA | -/- | < 1.75 inches |
| NY | Between approach slab and roadway | -/- |

# CHAPTER 3. FIELD INSPECTION OF APPROACH SLABS AND THE END OF BRIDGE

## 3.1 Field Inspection of Bridges

Field inspection and select testing of existing Iowa bridges with focus on the condition and performance of the approach slabs and the end of the bridge was conducted in June of 2018. WJE and the TAC committee of Iowa DOT selected and reviewed a total of ten bridges in counties located in eastern Iowa; Jefferson, Washington, Muscatine, and Lee. The time in-service of the bridges varied between two (2) years and twelve (12) years. The construction type of the bridges also varied with both integral abutment and stub abutment bridges included and the pavement type varied between portland cement concrete (PCC) pavement and hot-mixed asphalt (HMA) pavement. A summary of the preliminary list of bridges is shown in Table 3.1.

Based on the review and subsequent discussion with Iowa DOT, WJE selected 8 bridges for field inspections. Detailed field inspections were completed for four bridges while limited inspection (visual and borescope camera) was performed on four bridges. As shown in Table 3.1, the bridges were selected to cover three main variables: 1) abutment type, 2) type of roadway pavement, and 3) time in service.

*Table 3.1. List of Selected Bridges for Field Inspection.*

| Bridge | County | Year Built | Abutment | Approach Slab Details | Roadway | Inspection (Order*) |
|---|---|---|---|---|---|---|
| 5111.5O034 | Jefferson | 2006 | Stub | RK-20, RK21, RK-19B | PCC Pavement | Detailed (1) |
| 5126.5S078 | Jefferson | 2009 | Integral | RK-20, RK-19B | PCC Pavement | Detailed (2) |
| 5622.5O061 | Lee | 2011 | Integral | RK-20, RK-21, RK30 | PCC Pavement | Detailed (5) |
| 5624.2O061 | Lee | 2011 | Integral | RK-20, RK-21, RK30 | PCC Pavement | Detailed (8) |
| 9245.6S001 | Washington | 2015 | Stub | RK-20, RK-22 | HMA Pavement | Visual (3) |
| 5617.7L061 | Lee | 2011 | Integral | RK-20, RK-21, RK30 | PCC Pavement | Visual (7) |
| 5657.4O002 | Lee | 2011 | Integral | RK-20, RK-21, RK30 | PCC Pavement | Visual (4) |
| 5627.1O061 | Lee | 2011 | Integral | RK-20, RK-21, RK30 | PCC Pavement | Visual (6) |
| 9253.5S001 | Washington | 2016 | Integral | RK-20, RK-22 | HMA Pavement | None |
| 7066.0S006 | Muscatine | 2013 | Integral | RK-20, RK-21 | PCC Pavement | None |

*Order: Sequential order in which the bridges were inspected in the field

Detailed field inspections included visual inspection and documentation of the condition of surfaces of the bridge approach pavement, roadway shoulder, abutment wings, abutment footing face under the bridge, subdrain outlets for the abutment backfill drainage, berm slope erosion, and joint type and condition. They also included collection of elevation survey data to measure approach slab elevations with respect to the bridge, nondestructive evaluation of the access ports (if present and accessible) using a borescope, and Ground Penetrating Radar (GPR) scanning of the approach panels. Limited field inspections were conducted in a similar fashion with the exception that no elevation survey measurement or GPR scanning was performed. Description of the inspection methods used during this study is provided in the following sections.

### *3.1.1 Visual Survey*

A visual survey of the exposed and accessible surfaces of the approach slabs and related elements mentioned above was performed. For field documentation, WJE utilized an in-house annotation program (Plannotate 2) that was developed specifically to annotate digital inspection data and photographs onto PDFs using tablet computers. Inspection data included the location of potential voids and settlement, observed distress conditions, dimensional information of distress, and photographs of inspected elements and observed conditions. The findings of the visual assessment were assessed in conjunction with the GPR testing and elevation survey results.

Bridge components were assigned a condition rating between poor and good. A good condition means that the amount of distress was minor. A fair condition means that the distress was noticeable, but does not require maintenance. A moderate condition means that significant amounts of distress were observed and the component would be expected to require maintenance in the near future. A poor condition means that the component is at the end of its service life. The rating G to P means that the condition varied from good to poor within the one component.

### 3.1.2 Borescope (Snake Camera) Surveys

Iowa DOT had been incorporating access ports at the wing walls of bridges to investigate if settlement of the backfill beneath the approach slab has occurred adjacent to the bridge joint. The access ports consist of a PVC pipe installed near the bridge/approach slab joint and extending through the wing wall to the backfill under the approach slab. WJE used a borescope, also known as snake camera, to collect visual data inside the access ports to assess if settlement of the backfill had occurred. The borescope was also used to investigate the extent of voiding under the slabs where GPR testing and collected cores confirmed the presence of a void.

### 3.1.3 Elevation Surveys

A robotic total station was utilized to collect elevation survey data of selected portions of the approach slab for each bridge at which detailed inspections were completed. Survey data was typically collected for both lanes of the two approach slab panels leading to the bridge. The main purpose of the survey data was to determine if settlement of the approach slab had occurred relative to the bridge and to determine the Bridge Approach Performance Index (BI). Contour plots of the approach slab elevations were created using the survey data.

The Bridge Approach Performance Index (BI) was developed by White et al. (2005) to specifically assess the condition of bridge approaches. The BI value is calculated as the area between the original profile and the existing profile of the approach slab divided by its length. According to White et al. (2005), the original profile can be assumed as a straight line connecting the bridge slab to the pavement. The area is then calculated by integrating the original and existing profile over the slab length. White et al. (2015) state that high BI value is related to poor approach slab performance. During this study, three elevation lines were typically collected per lane. Therefore, the BI value was calculated for six discrete elevation lines for each bridge approach. The average of the six BI values was calculated and used to assess the condition of the bridge approach. It is noted that White et al. (2005) proposed assessment method uses the BI value in conjunction with IRI value to assess the condition of bridge approaches. As IRI measurements were not collected in this study, BI values were used to conduct a relative assessment based on the survey data.

### 3.1.4 Ground Penetrating Radar Testing

Ground penetrating radar (GPR) testing was conducted on each approach slab to assess abutment backfill settlement resulting in the presence of voiding beneath the approach. WJE has used this technique to identify voids under approach slabs during a previous project with Iowa DOT in western Iowa in 2011. The results of this project were reported to Iowa DOT in a report titled "Approach Slab Assessment - Full Testing Program Report" and dated November 2, 2011.

***Overview of GPR Test Method***

GPR is a geophysical nondestructive testing technique for the evaluation of structural elements and materials. The method utilizes electromagnetic waves to assess the internal characteristics of the material. GPR surveys performed on structural concrete elements allow for the detection and location of embedded objects (mild steel reinforcement, steel anchors, prestressing/post-tensioning strand, metal and plastic conduit), assessment of member thickness and element geometry, identification of larger internal conditions such as poor consolidation and flaws, and detecting presence of voids under slabs on grade such as approach slabs. Testing involves the use of a high-frequency radar antenna which transmits electromagnetic pulses along discrete scan lines at the surface of the structural element. The signals are reflected from material interfaces of differing dielectric properties along the propagation path of the waves. Signals are collected by the antenna, amplified, displayed and stored for subsequent interpretation. Antennae with different operating frequencies permit GPR surveying at various penetration depths. Additionally, post-processing software integrating signal filtering and visualization options allows for subsequent analyses of collected GPR scans.

***GPR Testing and Data Analysis Procedures***

GPR testing was performed using a Sir4000 GPR control unit manufactured by Geophysical Survey Systems, Inc. (GSSI) and a 2.6 GHz dipole antenna. A scanning cart specially designed for large scale bridge surveys was used to collect the data. Discrete scans were collected on the top surface of the approach slabs at a spacing of 2 feet on center. Each scan collected within the primary lanes was typically started several pavement joints from the bridge and extended past the bridge joint, thereby including several unreinforced pavement panels and the reinforced approach slab panels adjacent to the bridge. Additionally, GPR scans were collected on the shoulders within the concrete areas included in the approach system, which included one to three approach slab panels. Figure 3.1 shows a photograph of the GPR testing in progress while collecting a scan at the edge of an approach slab.

Data post-processing and analyses of the collected GPR scans were completed using software provided by GSSI, commercially known as *Radan* (Version 7.0). Data processing generally consisted of application of position correction, a series of finite impulse response (FIR) filters, and exponential gain adjustments intended to amplify and clarify the radar reflections from the bottom of the approach slab. A relative assessment of the amplitude of signal reflections from the bottom of the approach slab was made to identify areas where potential voiding under the slab exists. An example of a portion of a post-processed GPR scan is provided in Figure 3.2.The example scan was collected at the west approach of Bridge 5622.5O061 within the north shoulder and clearly shows a void beneath the approach panel extending more than 9 feet from the bridge abutment. The void is characterized by the high amplitude, negative (black) signal reflection from the bottom surface of the approach slab.

In order to verify the GPR results, concrete cores were drilled at select locations at each of the bridges where GPR data was collected and the presence of voids under the slabs was visually confirmed. Visual observations at each core locations were used calibrate the results of the GPR scanning and subsequent data interpretation.

*Figure 3.1. GPR testing in progress on an approach slab.*

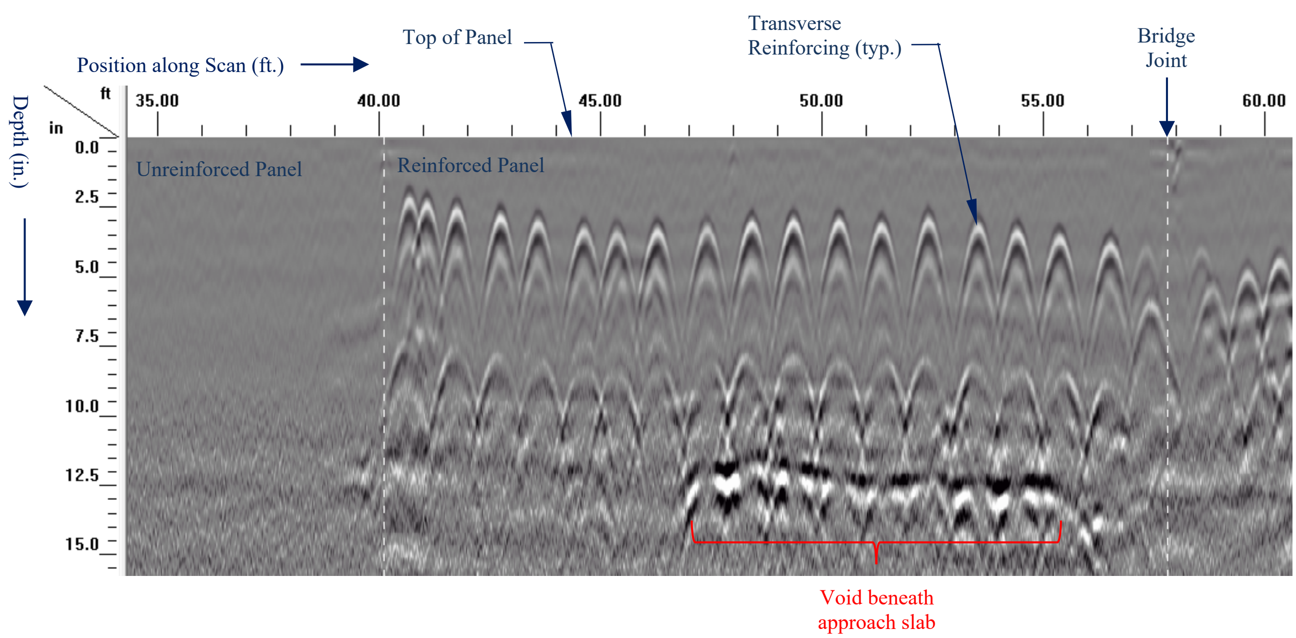



*Figure 3.2. Typical processed and filtered GPR scan indicating presence of void under the approach slab adjacent to the bridge joint. Example represents portion of Scan 07 collected at Bridge 5622.5O061, West approach; North shoulder.*

## 3.2 Field Inspections Results

Detailed and limited visual field inspections were conducted at each of the eight (8) bridges. During the visual inspection, condition ratings were assigned to each component of the bridge. Photographs of the features of each bridge are provided in Appendices A through H.

The joints between the approach slab and the bridge, between adjacent approach slab panels, and between the approach slab and the pavement were inspected. Iowa DOT Standard Road Plans show that different joints are used within each approach slab. Figure 3.3 provides schematics showing the type, location, and order of the joints in the approach slab with an abutting PCC pavement or an abutting HMA pavement. The joint between the approach slab and the bridge deck is a CF joint if an integral abutment is present and an E joint if a stub abutment is present. The CF joint width depends on the length of the bridge, as shown in Figure 3.4. Schematics for CF, E, CD, EF, B, DW, and RT joints are provided in Figure 3.4 through Figure 3.9. The following subsections provide the findings from the field investigation of each bridge.

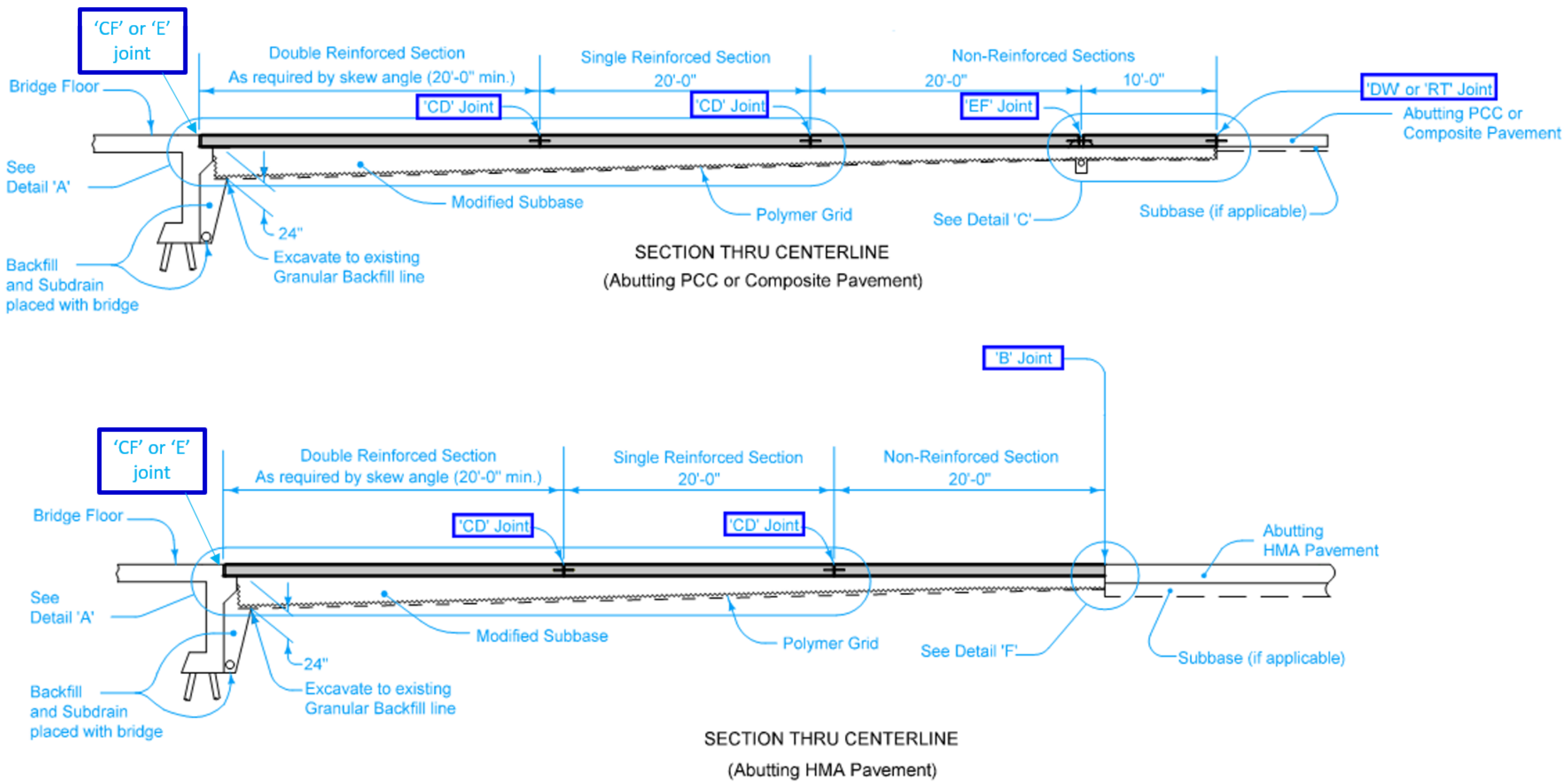
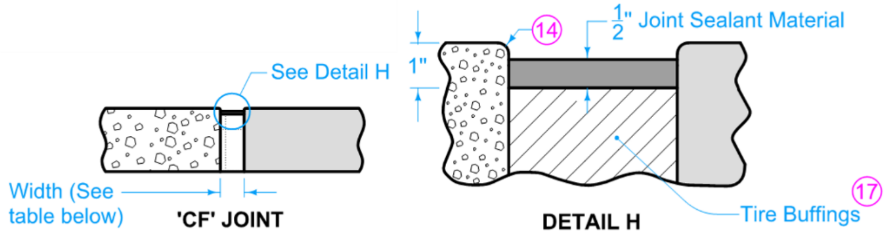



*Figure 3.3. Section views of the approach slabs from the Iowa DOT standard (IowaDOT Standard Road Plans).*




| TYPE | WIDTH |
|---|---|
| CF-1 | 2" |
| CF-2 | 2 1/2" |
| CF-3 | 3" |
| CF-4 | 3 1/2" |

| JOINT TYPE FOR MOVEABLE ABUTMENT BRIDGES | | |
|---|---|---|
| | Maximum Bridge Length | |
| Joint | Concrete Beam or Slab | Steel Girder |
| CF-1 | 370' | 250' |
| CF-2 | 465' | 320' |
| CF-3 | 575' | 400' |

*Figure 3.4. Schematic showing CF joint according to Iowa DOT standards (IowaDOT Standard Road Plans).*

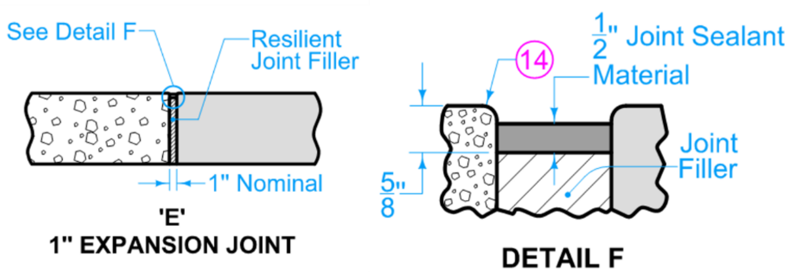



*Figure 3.5. Schematic showing E joint according to Iowa DOT standards (IowaDOT Standard Road Plans).*

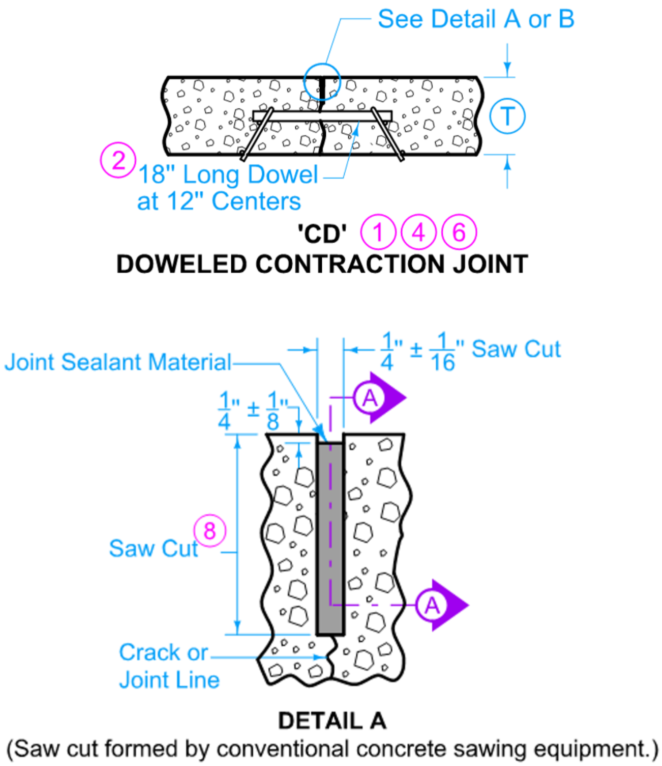



*Figure 3.6. Schematic showing CD joint according to Iowa DOT standards (IowaDOT Standard Road Plans).*

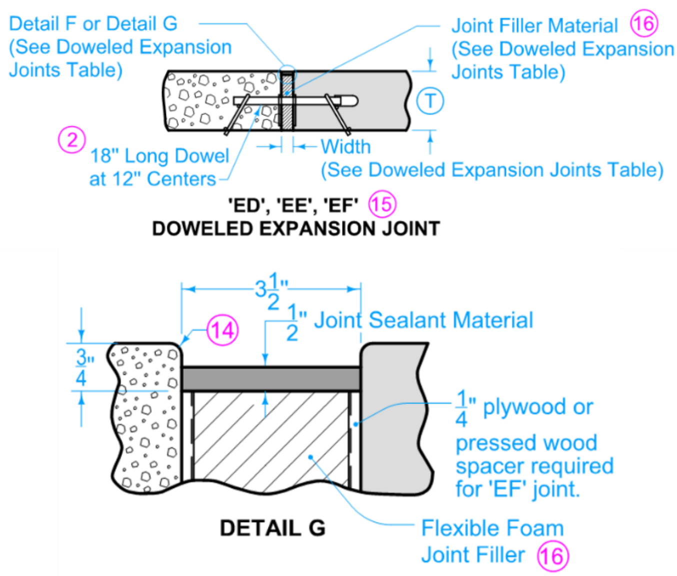



*Figure 3.7. Schematic showing EF joint according to Iowa DOT standards (IowaDOT Standard Road Plans).*

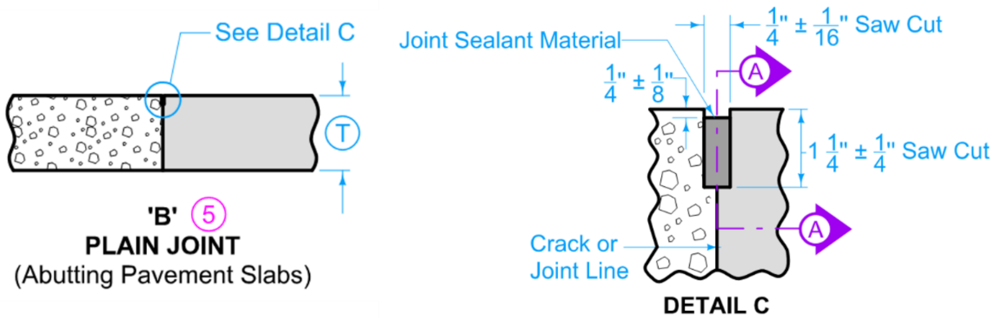



*Figure 3.8. Schematic showing B joint according to Iowa DOT standards (IowaDOT Standard Road Plans).*

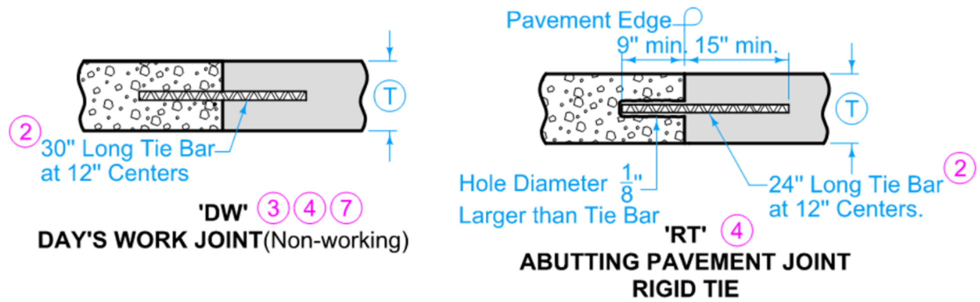



*Figure 3.9. Schematics showing DW and RT joints according to Iowa DOT standards (IowaDOT Standard Road Plans).*

### *3.2.1 Bridge 5111.5O034, Libertyville Rd over Route 34, Stub abutment, 2006*

Bridge 5111.5O034 is located in Jefferson County and carries traffic on Libertyville Rd over Route 34, approximately 1.5 mi west of junction IA #1. It was built in 2006 with a superstructure consisting of multiple steel stringers. The bridge has stub abutments and is abutting PCC pavement.

***Visual Observations***

This bridge is in good condition overall. Only a few components were in moderate condition and none were in poor condition. Photographs are provided in Appendix A. Table 3.2 describes the specific features and distresses observed.

*Table 3.2. Details from visual inspection of Bridge 5111.5O034.*

| **Approach** | | **East Approach** | **West Approach** |
|---|---|---|---|
| Pavement surfaces | | Minor surface damage present | Minor cracking at bridge joint<br>Pavement slabs were cracked |
| Joints | E - Abutment | 1.75-inch width at bridge joint; 1-inch width at back-joint<br>Deteriorated at shoulder<br>Filled with debris | 2-inch width at bridge joint; 1-inch width at back-joint<br>Looked intact<br>Filled with debris |
| | First CD | ----------- | ----------- |
| | Second CD | 5-inch spall present | ----------- |
| | EF - Pavement | 3.5-inch width<br>Sealant tore out easily<br>Vegetation present | 3.75-inch width<br>Edge raveling present |

***GPR, Cores, and Borescope***

GPR scanning results for the east and west approach are provided in Figure 3.10 and Figure 3.11, respectively. Each figure provides a plan view of the approach slab system. The dotted lines indicate the discrete GPR scans collected, generally in the direction toward the bridge. The scan lines have been color-coded to indicate either an unvoided condition (contact between bottom of slab and the base material, shown in blue) or a voided condition (shown in red). Additionally, the likely extent of voiding based on the discrete scan results is shown as a shaded red region. The results showed that voids beneath the slabs of this fixed abutment were relatively minor. The east approach had one void that extends 5 feet into the approach slab under the shoulder of the eastbound lane, and a narrow void that extends 9 feet into the approach slab on the shoulder side of the westbound lane. The west approach only had one void that extends 3 feet under the slab along the south barrier.

One core was collected on the east approach to verify the GPR results. The core was located close to the shoulder of the westbound lane in the east approach (Figure 3.10) and the pavement measured 14.5 inches in depth. Upon coring through the slab, the core sample dropped approximately 0.25 inches, as shown in Figure 3.12, indicating a 0.25 inch void at this location.

No inspection ports were found in the wing walls of this bridge; therefore, borescope inspection under the bridge approaches was not feasible.

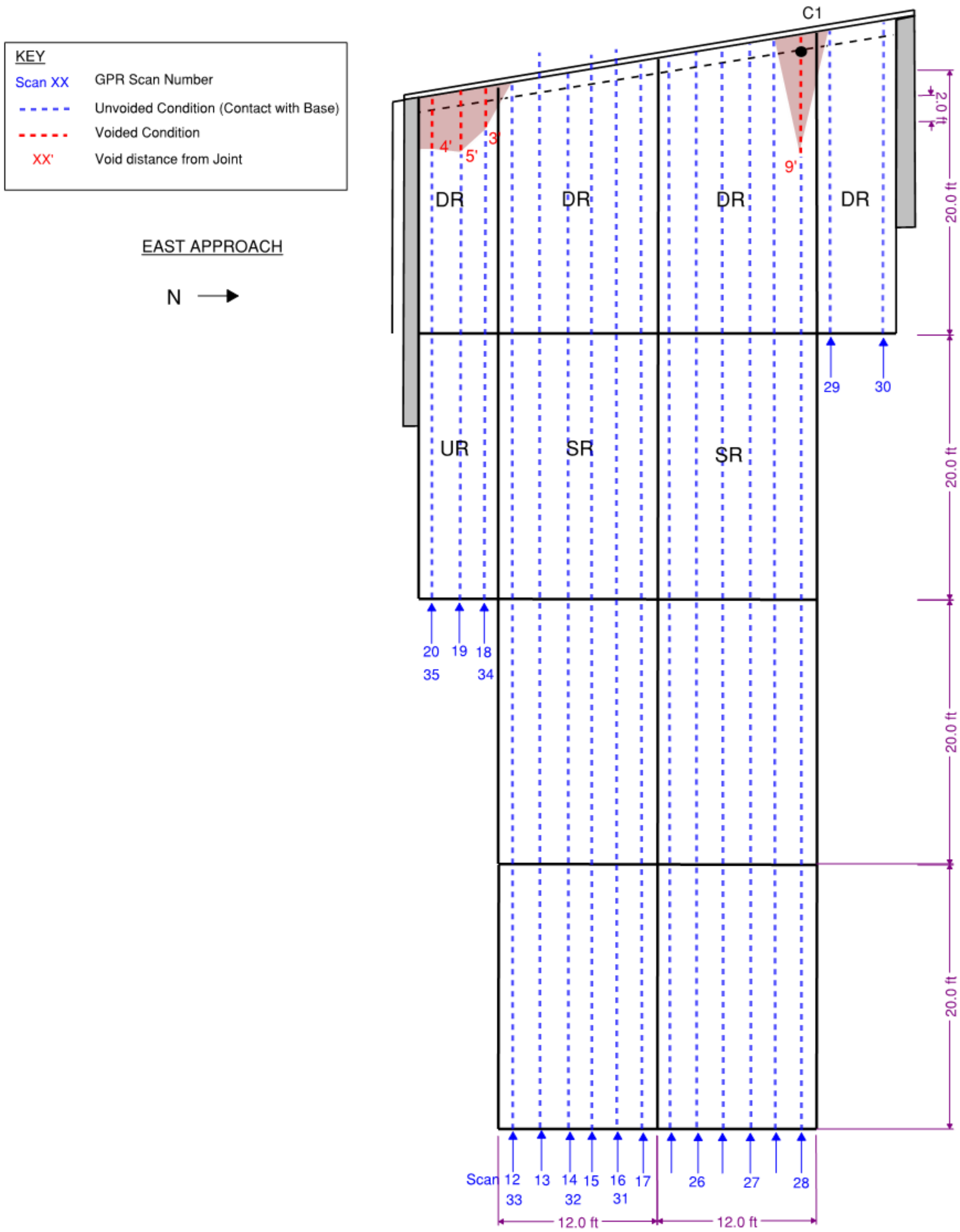



*Figure 3.10. GPR results and core location at east approach of Bridge 5111.5O034.*

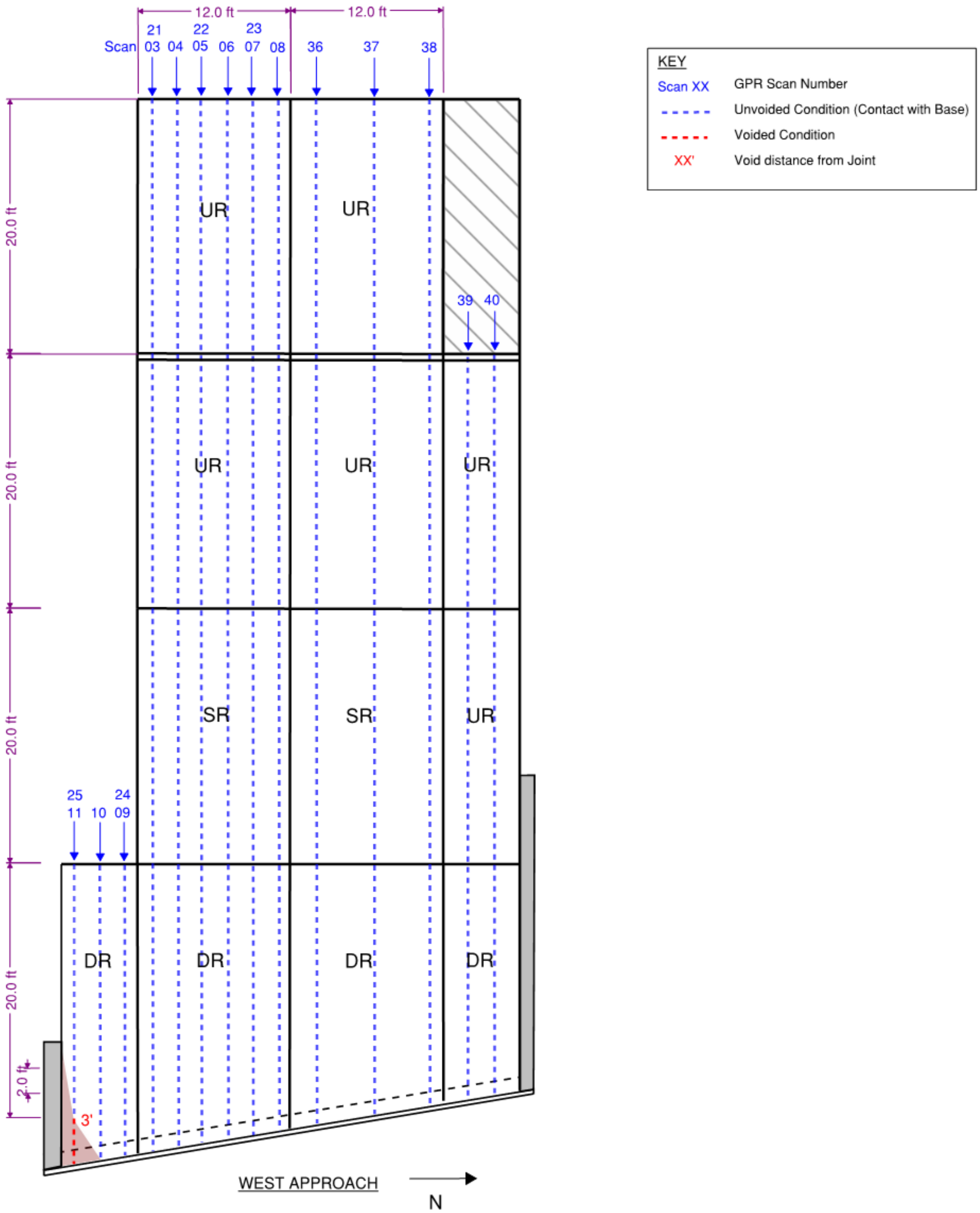



*Figure 3.11. GPR results at west approach of Bridge 5111.5O034.*


*Figure 3.12. View of core drop (1/4 inch) indicating presence of minor void under the slab.*

***Surveying***

The surface elevations of the bridge approaches were surveyed along three longitudinal lines in each lane, with only the approach lane surveyed per end. Figure 3.13 shows the contour maps for the approaches. All the elevations were measured relative to the elevation of the bridge joint. The depression of the slab was determined by assuming the intended profile was linear between the elevation of the bridge joint and the elevation of the roadway-end of the approach slab. Figure 3.14 shows the contour maps for the depression of the approaches. Figure 3.15 shows the depression visually in 3D surface maps. On the horizontal axes, the negative direction represents the distance away from the bridge. On the vertical axis of Figure 3.14, the negative direction represents north.

The depression maps show that negative depression (settlement) was present at the west approach while a slight positive depression was observed at the east approach. Note that this may be related to variation in the measurements. As shown in Figure 3.12, the pavement surface was grooved, which creates noise in the data and makes depressions and elevations smaller than the depth of the grooves insignificant. The survey data was used to calculate the BI value as 0.002 and 0.004 for the east and west approach, respectively. These values are small compared to values reported by White et al. (2005), which indicates that the bridge approaches are in good condition.

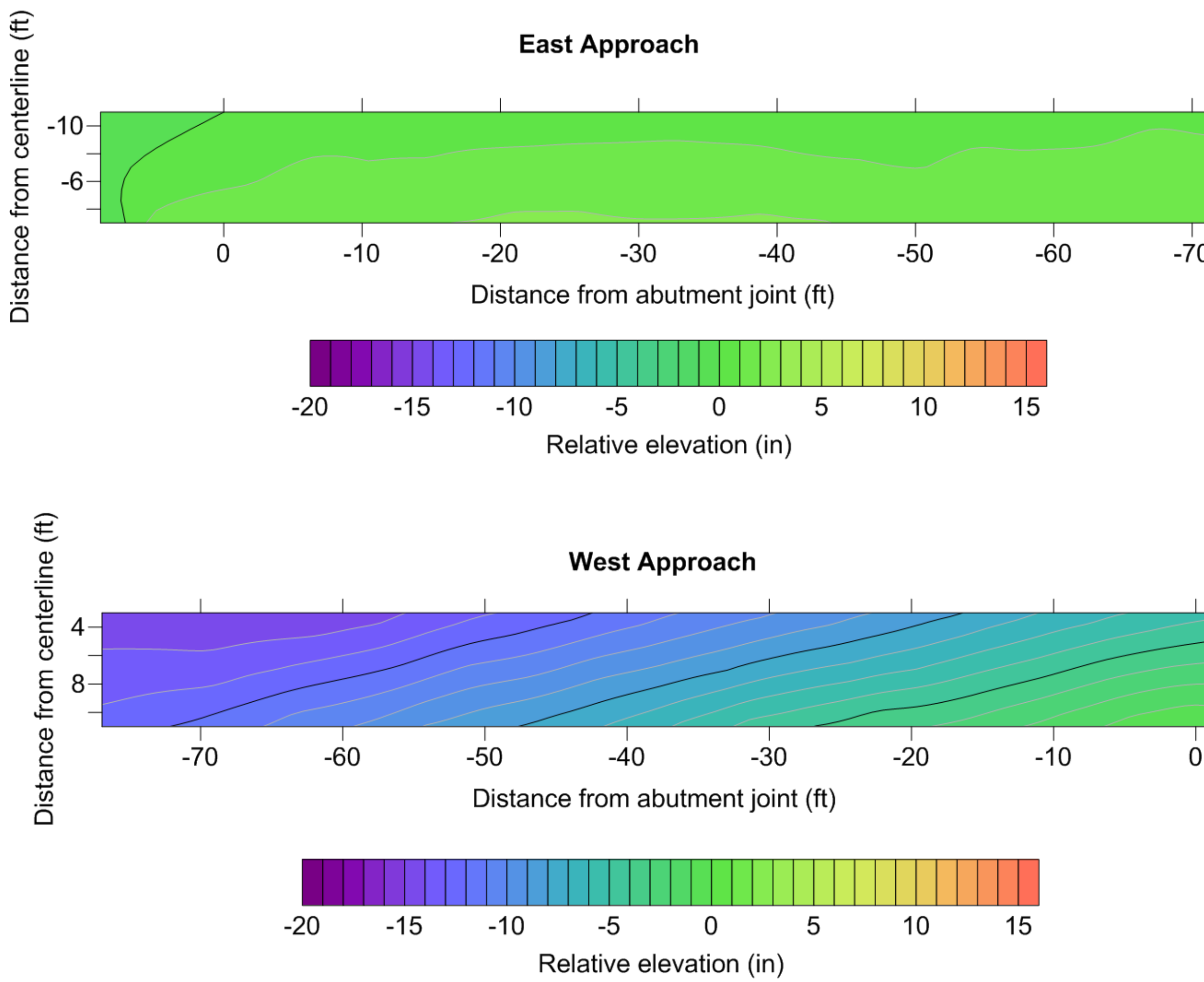



*Figure 3.13. Contour maps of the elevations of the approaches of Bridge 5111.5O034.*

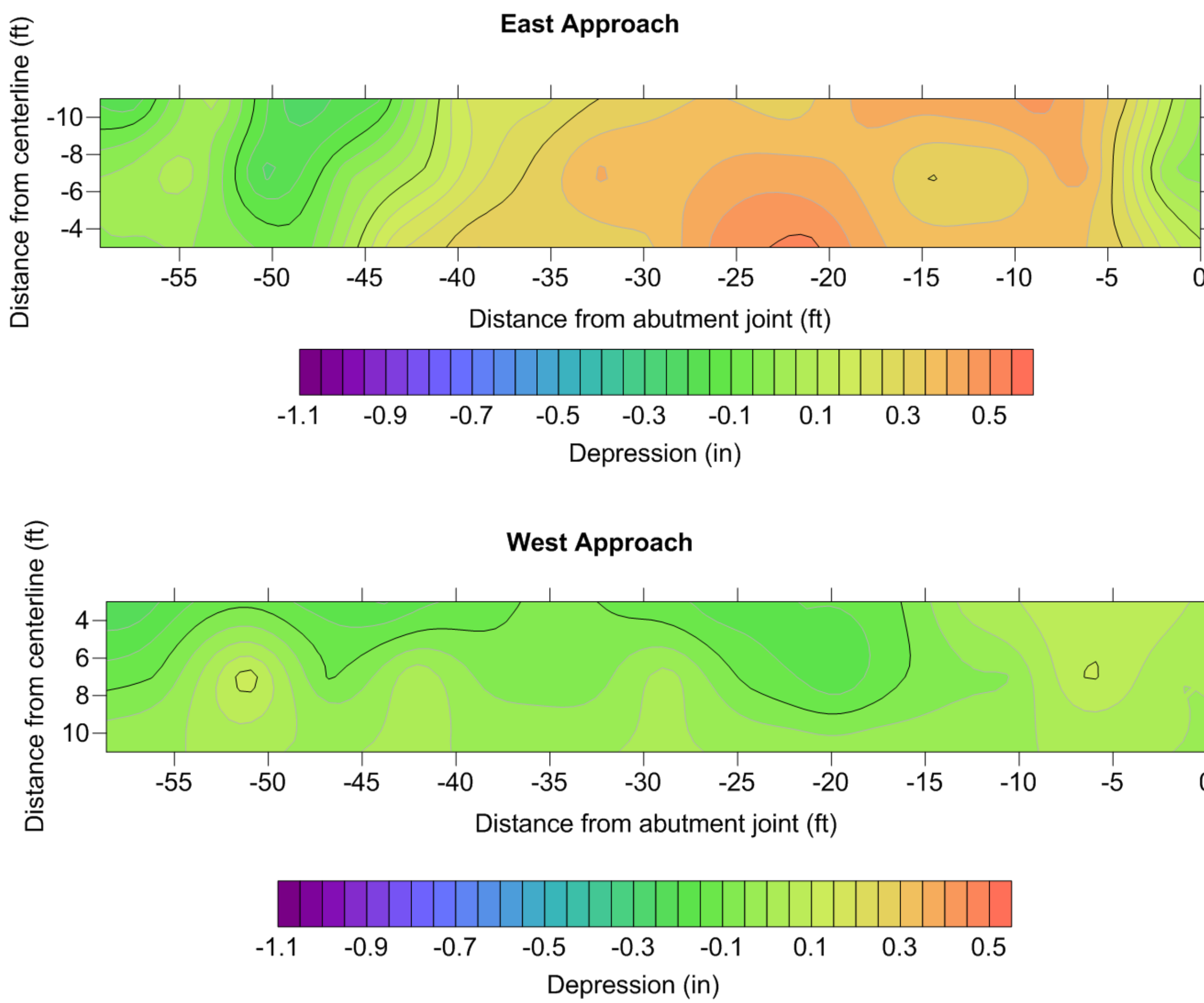



*Figure 3.14. Contour maps of the depression (net elevation) experienced by the approaches of Bridge 5111.5O034.*

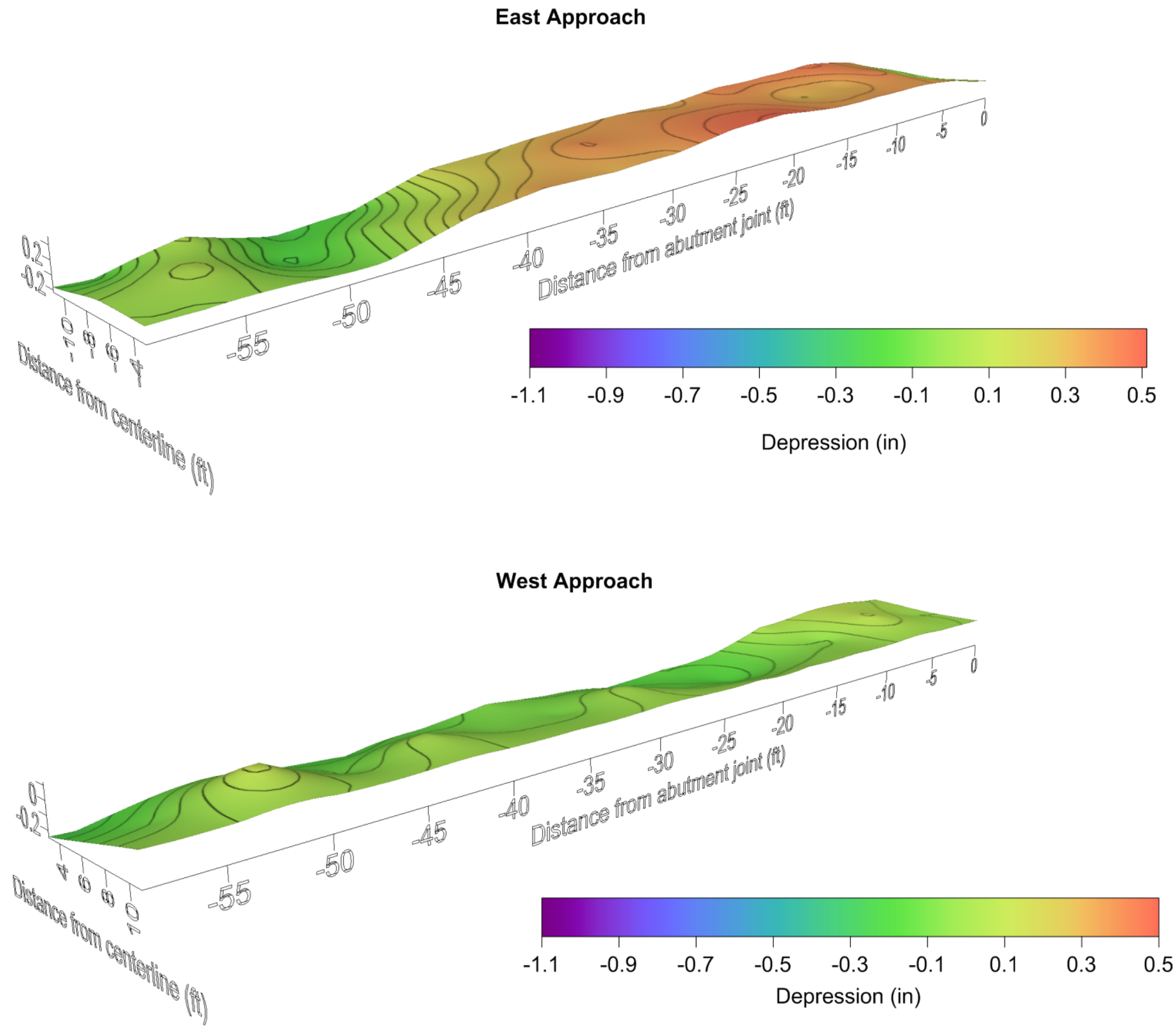



*Figure 3.15. Surface maps of the depression (net elevation) experienced by the approaches of Bridge 5111.5O034.*

### 3.2.2 Bridge 5126.5S078, Route 78 over the Skunk River, Integral abutment, 2009

Bridge 5126.5S078 is located in Jefferson County and carries traffic on Route 78 over the Skunk River, approximately 0.3 mi west of SR W47. It was built in 2009 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***

The approaches for this bridge are in moderate to fair condition. None of the components were in poor condition, but only the abutment wings at the west approach were in good condition. When approaching from the east, the bridge elevation was higher than the road elevation. Photographs are provided in Appendix B. Table 3.3 describes the specific features and distresses observed.

*Table 3.3. Details from visual inspection of Bridge 5126.5S078.*

| Approach | | East | West |
|---|---|---|---|
| Pavement surfaces | | Longitudinal crack present in second slab in westbound lane | Transverse cracks present in third and fifth slabs across full width of pavement |
| Joints | CF3-Abutment | 2.5-inch width<br>Sealant intact | Width varied from 2.5 to 3 inches<br>Filled with debris in eastbound lane<br>Impact sealing damage present |
| | First CD | ----------- | ----------- |
| | Second CD | ----------- | Minor raveling present |
| | EF - Pavement | Width varied from 2.25 to 3.25 inches<br>Sealant lost bond with pavement<br>Some raveling present | 2.5-inch width<br>Sealant was torn<br>Some vegetation, raveling, and spalling present |
| | DW or RT | ----------- | In good condition |
| | Contraction | ----------- | Some raveling present |
| Berm slope | | ----------- | No erosion; rip-rap in place |

***GPR, Cores, and Borescope***

GPR scanning results for the east and west approach are provided in Figure 3.16 and Figure 3.17, respectively. Both sets of GPR data show that the approach slabs had voids underneath them at the joint with the bridge deck and that the extent of the void into the approach slab increased with distance from the centerline. At the east approach, the void extends approximately 6 feet into the approach slab at the centerline and up to 13 feet into the approach slab at the barriers. At the west approach, the void extends approximately 4 feet into the approach slab at the centerline and up to 13 feet into the approach slab at the barriers.

Two cores were collected at this bridge, one core at each bridge approach. Core 1 is located in the westbound lane at the east approach (Figure 3.16). The pavement core measured approximately 13 inches in depth. The core dropped a distance of 3.5 inches confirming the presence of a large void at this location, as seen in Figure 3.18. Core 2 is located at the corner of the eastbound lane of the west approach, at the shoulder and the bridge deck. An epoxy-coated rebar was encountered during cutting of the core. A drill was used to reach the bottom of the slab where a void measuring 2.5 inches in depth was observed, as shown in Figure 3.19.

The borescope was inserted through the void underneath Core 1, the access ports at the west approach, and the northern access port at the east approach. It was not possible to inspect the southern access port because it was blocked by a wasp nest. The videos captured at the north access ports of the east and west approaches confirm the existence of voids underneath both approach slabs. For the west approach south access port the video was inconclusive as the port was filled by soil/backfill. Figure 3.20 provides an image from the video showing exposed rebar between the east approach slab and the backfill material behind the abutment wall. In addition, the access ports were mostly blocked by eroded, fine soil, as shown in Figure 3.21.

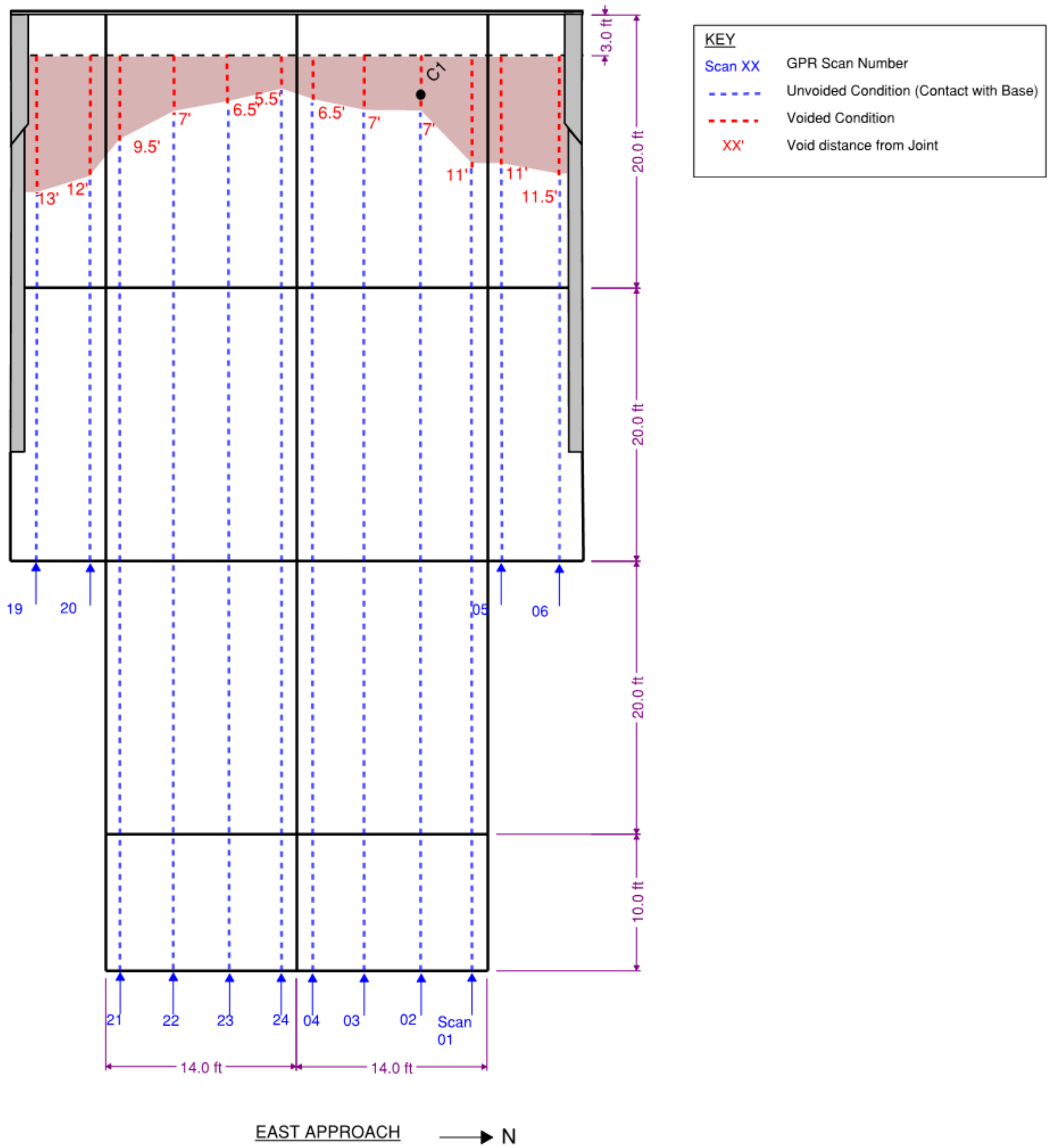



*Figure 3.16. GPR results and core location at east approach of Bridge 5126.5S078.*

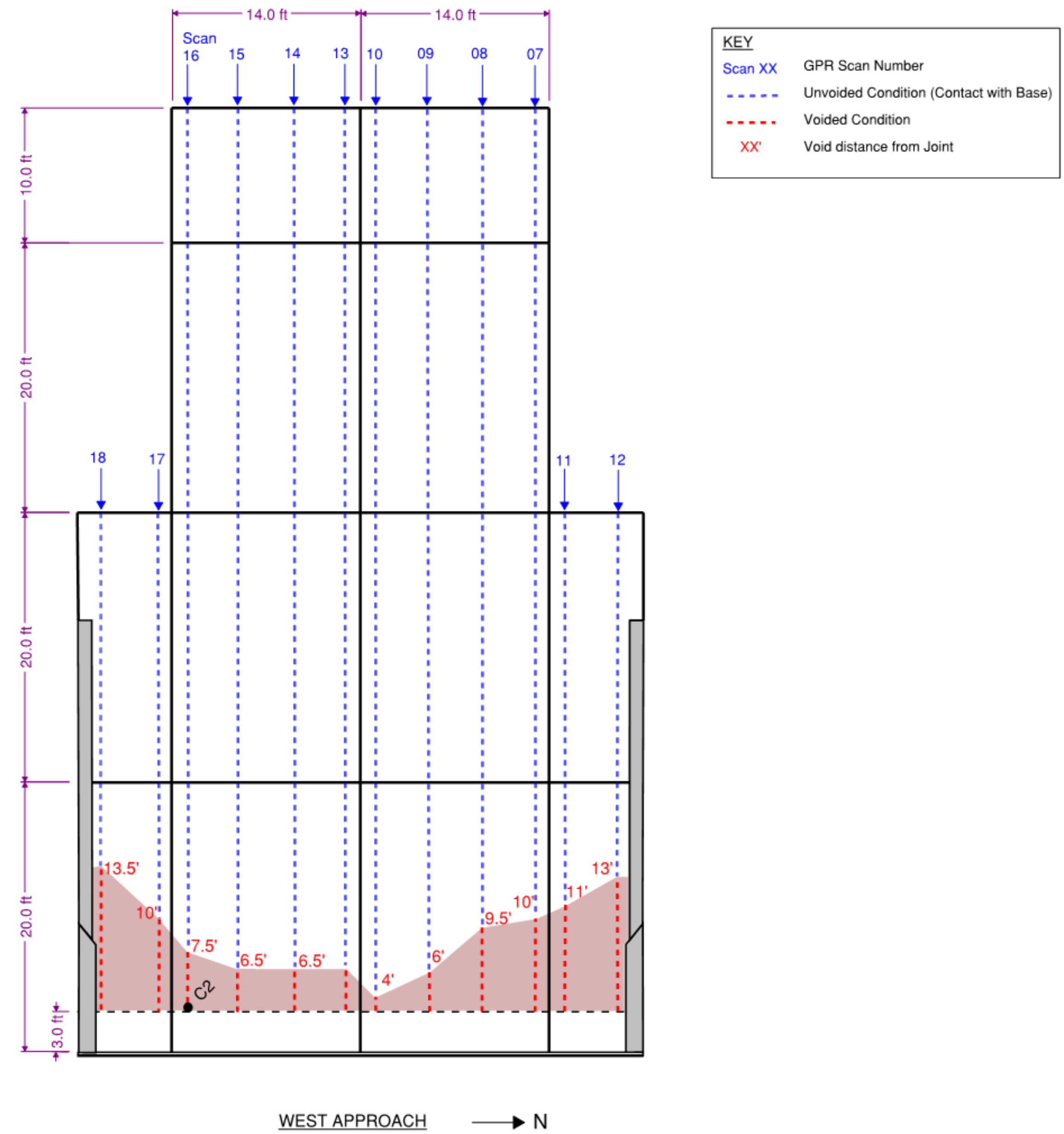



*Figure 3.17. GPR results and core location at east approach of Bridge 5126.5S078.*

*Figure 3.18. Measured drop (3.5 inches) of Core 1 of Bridge 5126.5S078.*

*Figure 3.19. Epoxy-coated rebar and voids found under Core 2 of Bridge 5126.5S078.*

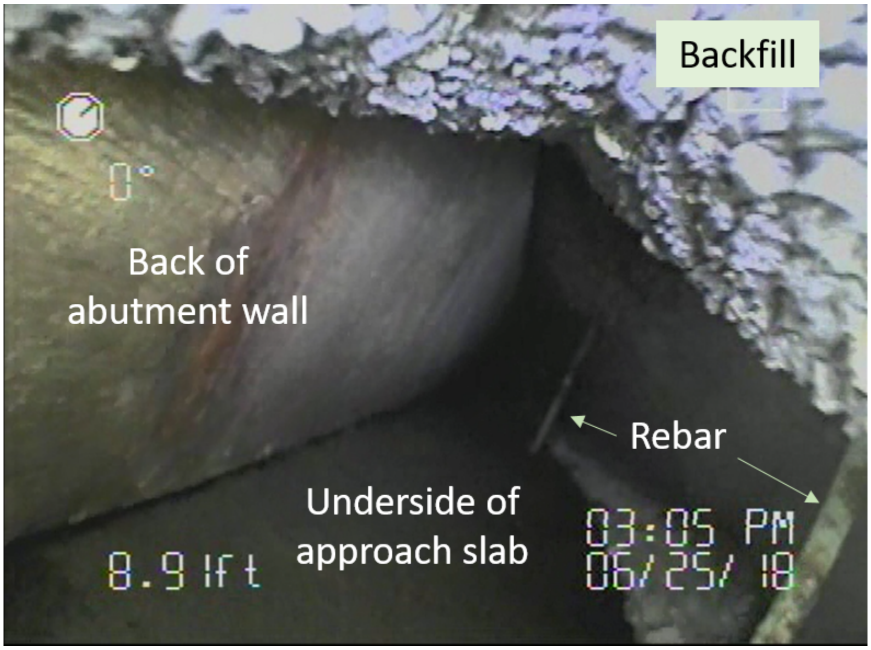



*Figure 3.20. View of void beneath east approach slab of Bridge 5126.5S078 from borescope.*


*Figure 3.21. View of debris in northern access port at the east approach of Bridge 5126.5S078.*

***Surveying***

The elevations of the bridge approaches were surveyed along two to three longitudinal lines in each lane and the depression (net elevations) was determined following the same procedure as for Bridge

5111.5O034. Figure 3.22 shows the contour maps for the approaches. Figure 3.23 shows the contour maps for the depression of the approaches and Figure 3.24 shows the data in 3D surface maps. On the horizontal axes, the negative direction represents the direction away from the bridge. On the vertical axis of Figure 3.23, the negative direction represents north.

The V-shaped patterns on the elevation maps show the drainage control. There is a slight deviation at the north sides of the bridge joints, but otherwise the pattern is predictable. The depression maps show that the maximum depression was 0.8 inches at the west approach and 0.35 inches at the east approach. The survey data was used to calculate the BI value as 0.016 to 0.030 for the east and west approach, respectively.

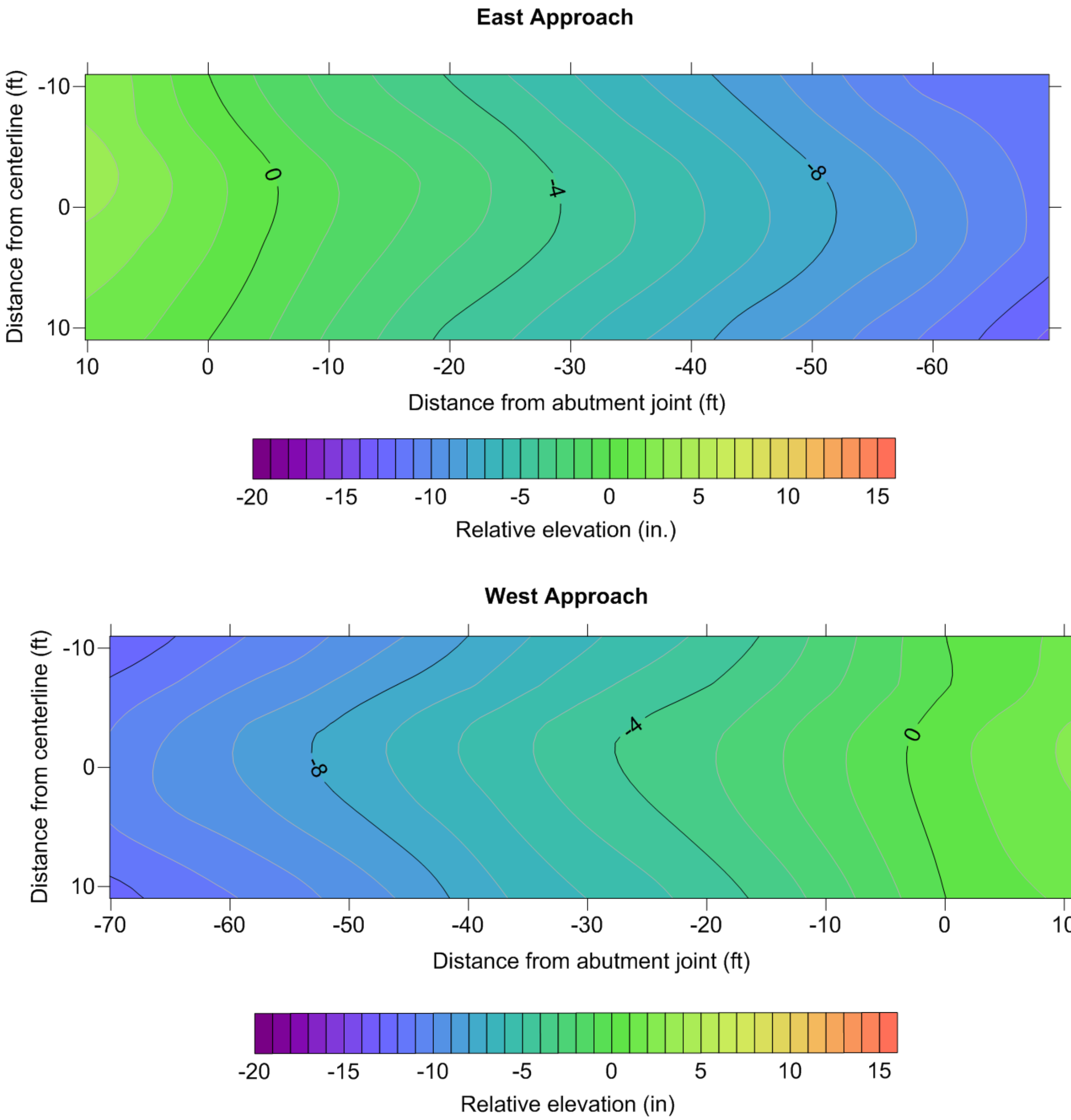



*Figure 3.22. Contour maps of the elevations of the approaches to Bridge 5126.5S078.*

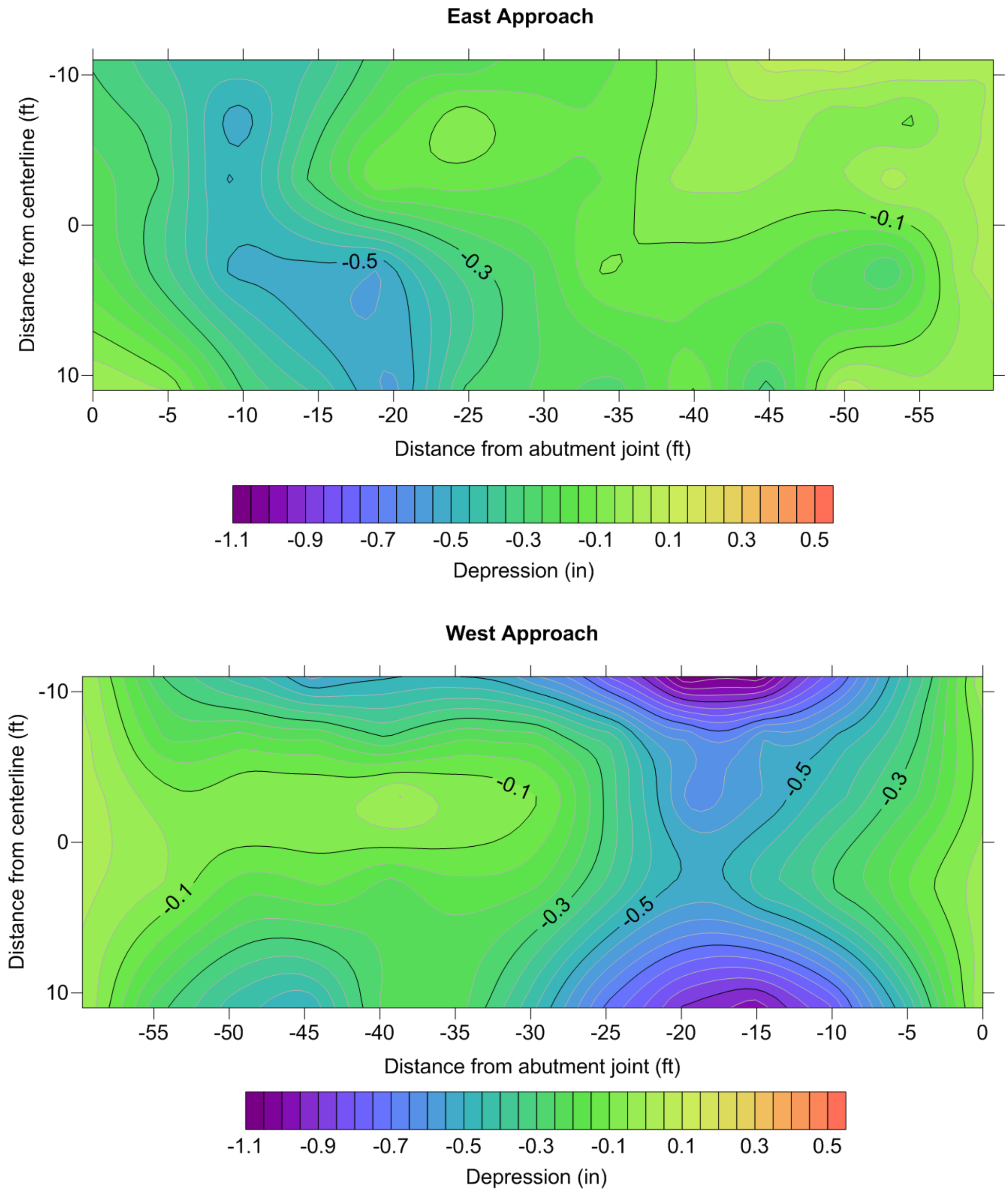



*Figure 3.23. Contour maps of the depression experienced by the approaches to Bridge 5126.5S078.*

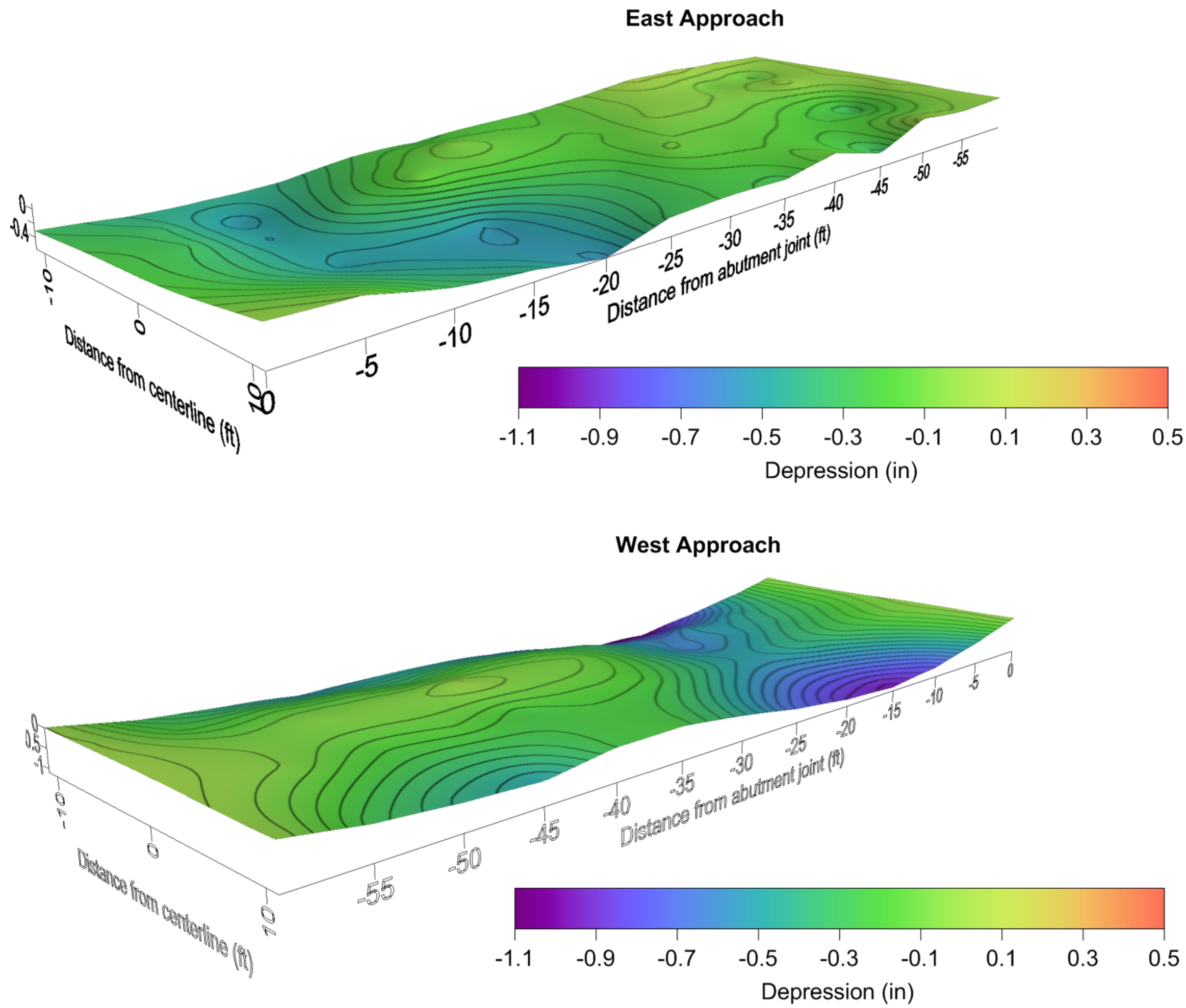



*Figure 3.24. Surface maps of the depression experienced by the approaches to Bridge 5126.5S078.*

### 3.2.3 Bridge 5622.5O061, J40 over Route 61, integral abutments, 2011

Bridge 5622.5O061 is located in Lee County and carries traffic on J40 over Route 61. It was built in 2011 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***

Conditions of this bridge varied from good to poor. The barriers were in poor condition at both approaches and the first CD joint in the east approach and the final contraction joint in the west approach were in poor condition as well. The west approach was in noticeably better condition than the east approach and when driving across the bridge, the abutment joint in the east approach had a slight bump while the west approach was smooth. Photographs are provided in Appendix C. Table 3.4 describes the specific features and distresses observed.

*Table 3.4. Details from visual inspection of Bridge 5622.5O061.*

| **Approach** | | **East** | **West** |
|---|---|---|---|
| Pavement surfaces | | No obvious cracks | |
| Joints | CF1 - Abutment | 3.5-inch width<br>Joint sealant completely deteriorated at some sections<br>Full with vegetation at some parts | 3.5-inch width<br>Deteriorated joint sealant<br>Lots of vegetation present |
| | First CD | 0.5-inch width<br>Sealant failed such that joint is see-through in some sections | Failed at several locations<br>Some raveling and chipping |
| | Second CD | 0.5-inch width<br>Sealant failed such that joint is see-through in some sections | Some deterioration and cracking in sealant |
| | EF - Pavement | Width varied from 1.5 to 2.5 inches<br>Minor cracking in sealant<br>Some vegetation present | Filled with rubber and mostly level<br>2.5-inch width<br>Minor cracking on top surface of sealant<br>Some debonding on edges at shoulder slab |
| | DW or RT | Minor cracking in sealant | ----------- |
| | Contraction | Deteriorated and cracked<br>Failed near shoulder | Joint has completely failed<br>Remnants of sealant left |
| Shoulder | | Ends after first slab | Ends three slabs after bridge<br>Curbs appear to have been sawed or chipped off |
| Barrier | | South: 2.5-inch gap<br>North: curb was cut out, leaving void | South: failure<br>North: cut joint with about 1-inch gap<br>Erosion at ends of both barriers |
| Berm slope | | No erosion | No erosion |
| Subdrain outlets | | Clear | Clear |

***GPR, Cores, and Borescope***

GPR scanning results for the east and west approach are provided in Figure 3.25 and Figure 3.26, respectively. Similar to Bridge 5126.5S078, the voids extend further into the approach slabs at the barriers or edge of pavement, than at the centerline of the pavement. At the east approach, the void extends 10 to 11 feet into the approach slab at the barriers and approximately 4 feet for the rest of the pavement. Similarly, the void extends 9 to 13 feet into the approach slab at the barriers of the west approach and extends approximately 4 feet at the centerline.

Four cores were collected at this bridge to confirm the GPR results, as shown in Figure 3.25 and Figure 3.26, with one core intentionally located in an area where GPR did not indicate the presence of voids. Core 1 was located near the abutment joint in the westbound lane of the east approach. The core was approximately 13 inches long and dropped 0.5-inch after being cut, as seen in Figure 3.27. The bottom of the core had large gravel and a cohesive, wet sand attached. Core 2 was located in the same slab as Core 1 but was about halfway between the abutment joint and the first CD joint in an area where GPR data did not indicate the presence of a void. As shown in Figure 3.28, no void or drop in the core was measured at this location. Similar to Core 1, large gravel and cohesive sand was attached to the bottom of the core which measured 13 inches in length. Core 3 was located in the westbound lane of the west bridge approach. The core was 12 inches long and had a void of 1.5 inches underneath it, as shown in Figure 3.29. Core 4 was located in the shoulder of the eastbound lane of the east approach, close to the first CD joint. The core length was 13 inches and had a void of 2 inches underneath it, as shown in Figure 3.30.

The borescope was inserted through Cores 3 and 4 and all of the access ports located in the wing walls. The back of the abutment wall was visible when the borescope was inserted through the cores, confirming the voids found by the GPR. Both of the access ports at the east approach and the north access port under the west approach were blocked with large aggregates such that the borescope could not get through. The south access port under the west approach had some metal across its entrance under the slab. The vertical rebar and bottom of the approach slab were visible, as shown in Figure 3.31.

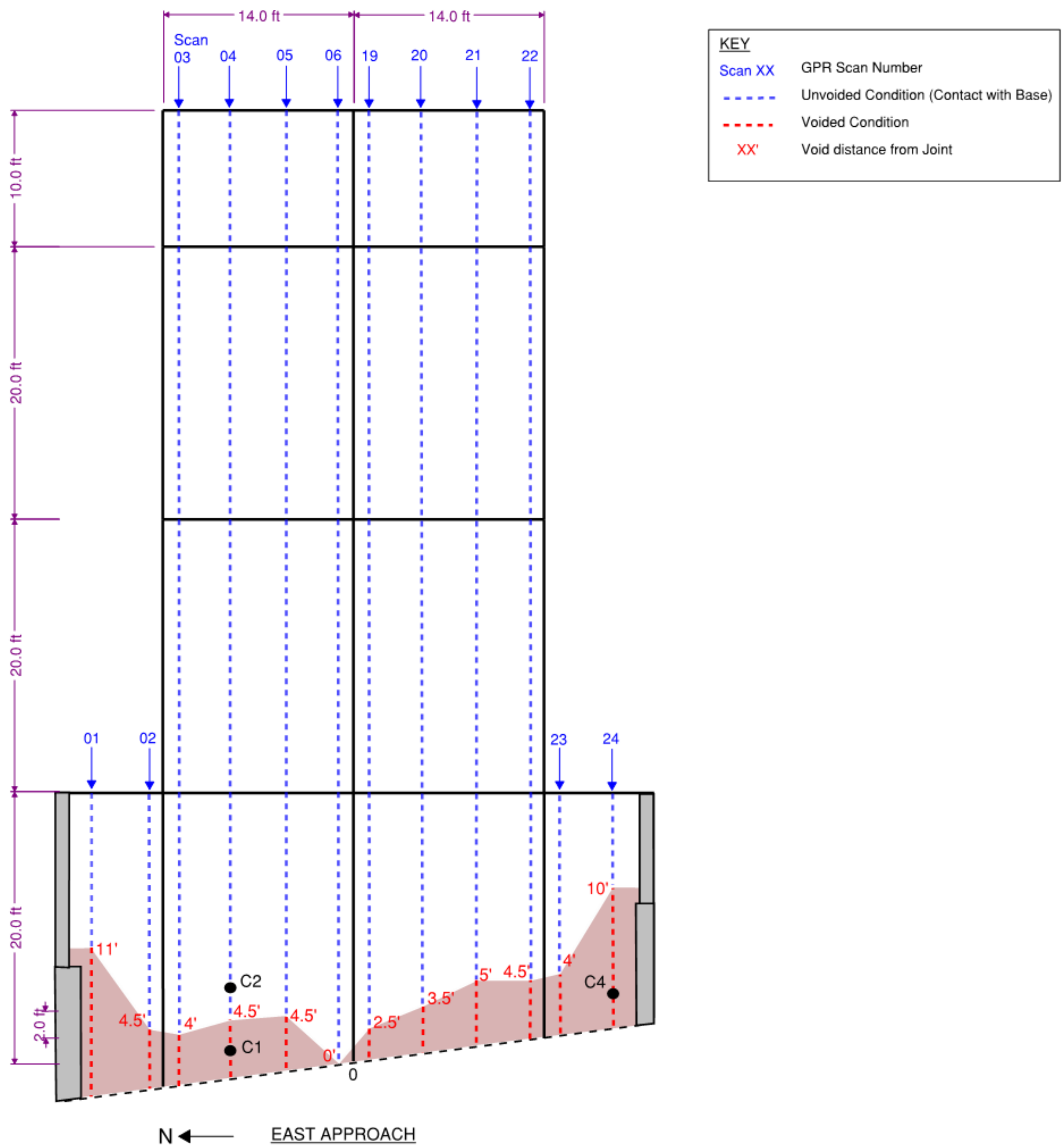



*Figure 3.25. GPR results and core locations at east approach of Bridge 5622.5O061.*

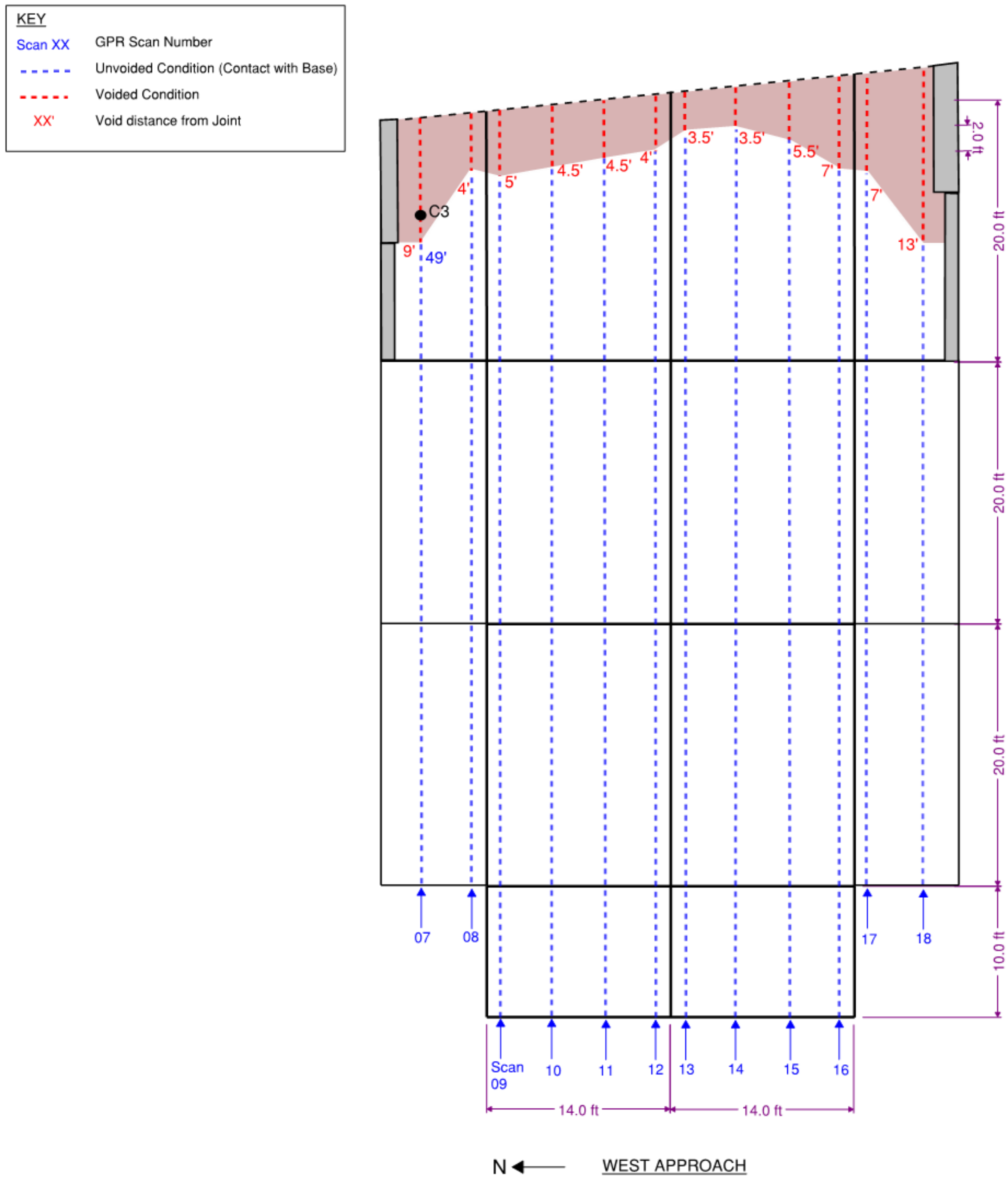



*Figure 3.26. GPR results and core locations at west approach of Bridge 5622.5O061.*

*Figure 3.27. Measured drop (0.5inch) of Core 1 of Bridge 5622.5O061.*

*Figure 3.28. View of Core 2 of Bridge 5622.5O061 (no void).*

*Figure 3.29. Measurement of void (1.5inch) underneath Core 3 of Bridge 5622.5O061.*

*Figure 3.30. Measurement of void (2.0inch) underneath Core 4 of Bridge 5622.5O061.*


*Figure 3.31. Entrance of south access port under west approach of Bridge 5622.5O061.*

***Surveying***

The elevations of the approaches were surveyed and the depression (net elevation) was determined following the same procedure as for Bridge 5111.5O034. Figure 3.32 shows the contour maps for the approaches and Figure 3.33 and Figure 3.34 show the depression in contour maps and 3D surface maps, respectively.

The V-shaped patterns on the elevation contour maps show the typical drainage pattern. For this bridge, the maximum amount of depression was 1.0 inches at the west approach and 0.6 inches at the east approach. The maps indicate that there are no areas with a bump at either approach. The BI value was calculated as zero and 0.014 for the east and west approaches, respectively. These values are small compared to values reported by White et al. (2005).

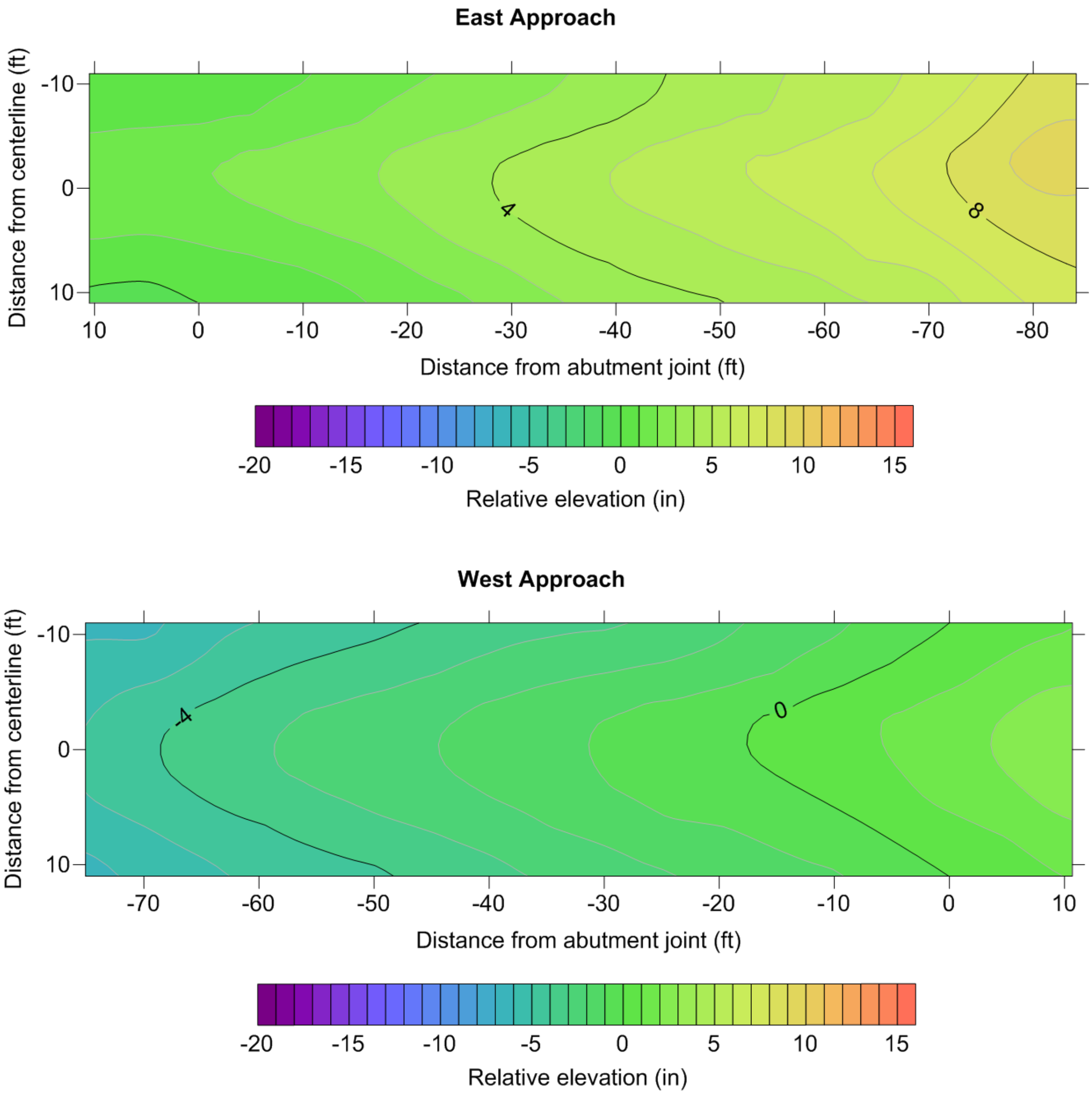
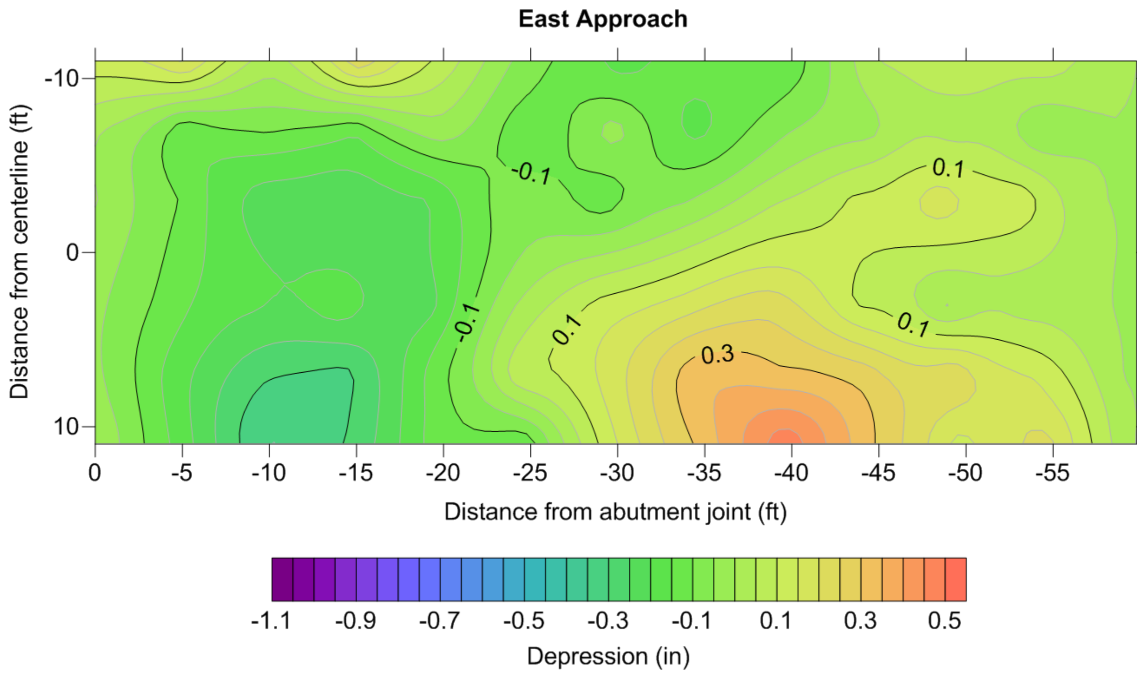



*Figure 3.32. Contour maps of the elevations of the approaches to Bridge 5622.5O061.*



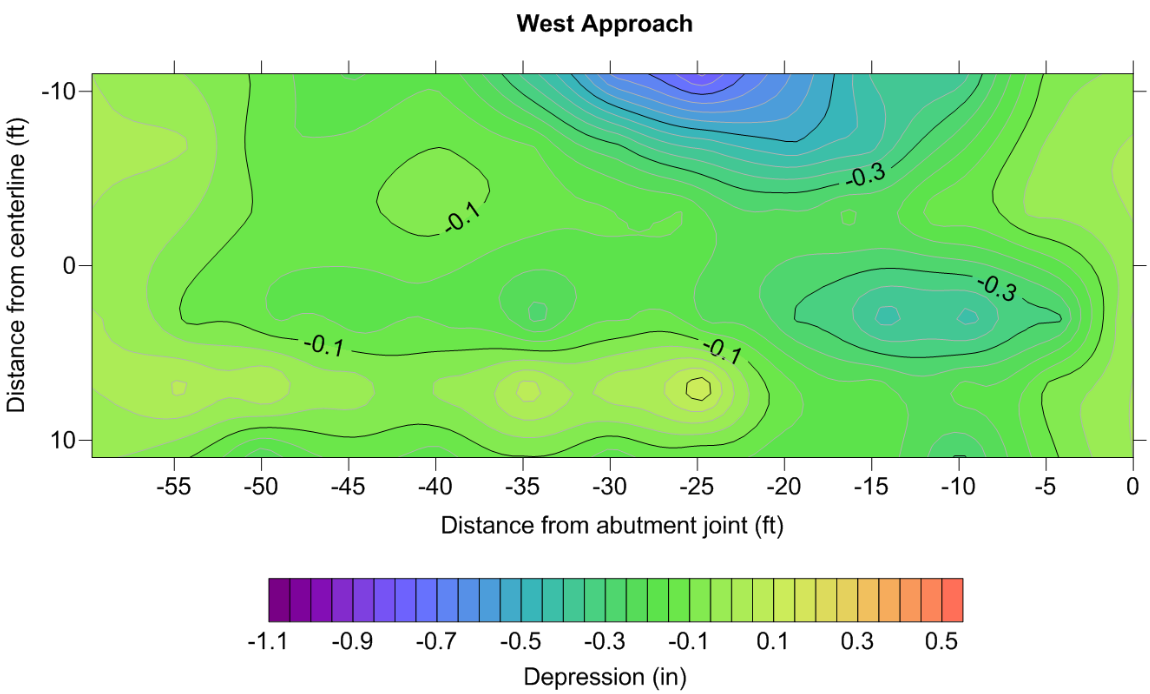




*Figure 3.33. Contour maps of the depression experienced by the approaches to Bridge 5622.5O061.*

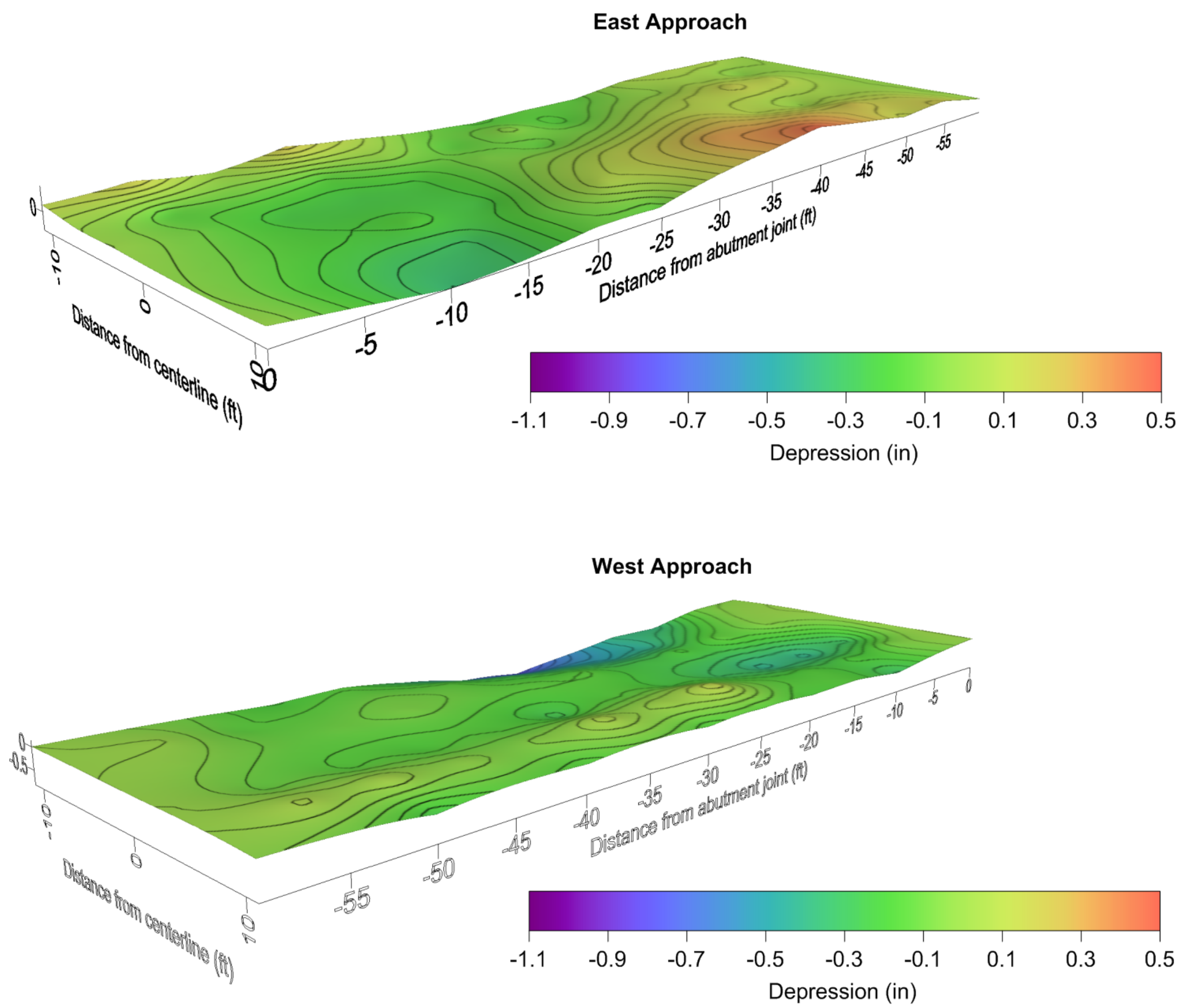



*Figure 3.34. Surface maps of the depression experienced by the approaches to Bridge 5622.5O061.*

### *3.2.4 Bridge 5624.2O061, 303rd Ave over Route 61, integral abutment, 2011*

Bridge 5624.2O061 is located in Lee County and carries traffic on 303rd Ave over Route 61. It was built in 2011 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***

While the substructure appeared to be in good condition, many of the joints were in a poor condition or had conditions varying from fair to poor. Photographs are provided in Appendix D. Table 3.5 describes the specific features and distresses observed.

*Table 3.5. Details from visual inspection of Bridge 5624.2O061.*

| Approach | | East | West |
|---|---|---|---|
| Pavement surfaces | | ----------- | Transverse cracking present in eastbound lane |
| Joints | CF1-Abutment | 2-1/8-inch width<br>Large portions of sealant were failed<br>Remnants of the sealant were cracked<br>Vegetation present in shoulders | 2.75-inch width<br>Sealant recessed up to 1.5 inches<br>No sealant present |
| | First CD | 0.5-inch width<br>Large portions of sealant were failed<br>Remnants of the sealant were cracked | 0.5-inch width<br>Sealant had deteriorated<br>Some portions had failed |
| | Second CD | 0.75-inch width<br>Sealant had either failed or separated from the slab | 0.5-inch width<br>Some portions had failed<br>Cracking and spalling around joint<br>Corner crack present |
| | EF - Pavement | 2-inch width<br>Large portions of sealant had failed<br>Raveling and spalling present | 1.25-inch width<br>Sealant settled and cracked<br>Deterioration and spalling around joint |
| | DW or RT | No sealant present<br>Very tight joint | No sealant present<br>Spalling present |
| | Contraction | 1-inch width<br>Large portions of sealant separated from slab<br>Some areas had failed | 0.75-inch width<br>Sealant was cracked and had settled<br>Sealant had completely separated from slab |
| Shoulder | | Ended after first slab | Ended after third slab |
| Barrier | | South: gap present between barrier and slab | North: gap present between barrier and slab |
| Berm slope | | No erosion<br>Some vegetation present | No erosion<br>Some vegetation present |

***GPR, Cores, and Borescope***

GPR scanning results for the east and west approach are provided in Figure 3.35 and Figure 3.36, respectively. Similar to other bridges, the voids extend further into the slab at the barriers than under the traffic lanes. On the westbound side of the east approach, the void extends 6 feet on the shoulder and then only 2 to 3 feet in the lane. There is no void roughly at the centerline of the pavement. On the eastbound lane of the east approach, the void extends 10 feet at the barrier and 3 to 4 feet in the lane. On the eastbound lane of the west approach, the void extends about 11.5 feet along the barrier and 3 to 4 feet under the traffic lanes. On the westbound side of the west approach, the void extends 9 feet along the barrier and about 3 feet under the traffic lanes.

Three cores were collected at this bridge. Core 1 was located at the middle of the shoulder of the eastbound lane of the west approach. It was approximately 12 inches long and had a void of 0.5 inches below it. The bottom of the core, shown in Figure 3.37, shows the large gravel attached to it. Core 2 was located near the centerline of the bridge close to the abutment joint of the west approach. The core was approximately 11 inches long and dropped a depth of 6.25 inches, as shown in Figure 3.38, confirming the presence of a sizable void. Core 3 was located in the westbound lane of the east approach, close to the abutment joint. The core was approximately 11.75 inches long and dropped about 3.25 inches, as shown in Figure 3.39.

The borescope was inserted into Core 2 and each of the access ports. Both of the access ports underneath the east approach were blocked by soil/ backfill. The south access port under the west approach was partially blocked by soil/backfill such that the borescope could not get through. However, the borescope successfully reached the void under the west approach through Core 2 and the north access port. The paving notch appears to be damaged, there is a gap between the approach slab and the paving notch, and extensive erosion has occurred, as shown in Figure 3.40.

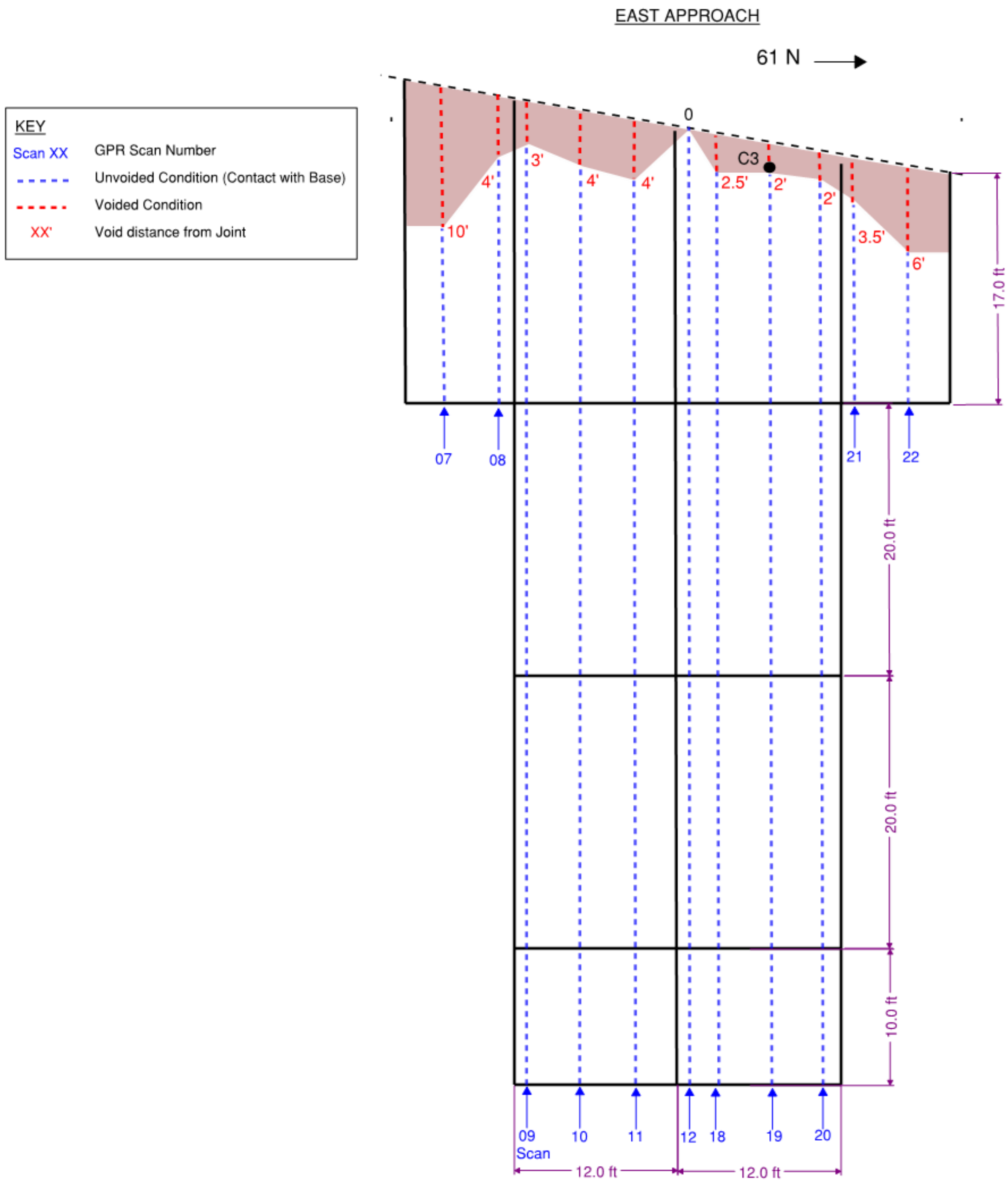



*Figure 3.35. GPR results and core locations for east approach of Bridge 5624.2O061.*

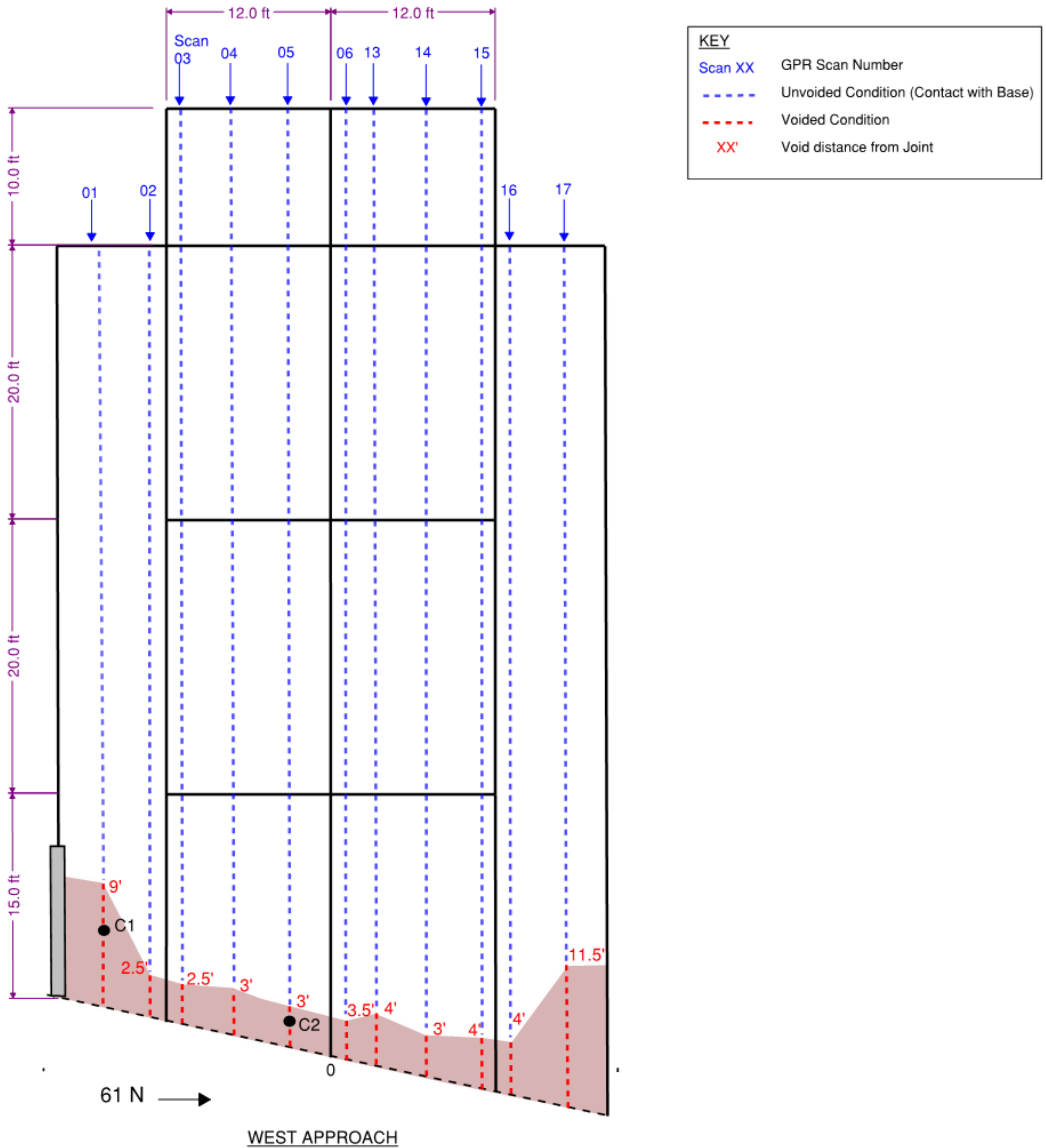



*Figure 3.36. GPR results and core locations for west approach of Bridge 5624.2O061.*

*Figure 3.37. Bottom of Core 1 of Bridge 5624.2O061, showing original contact with the base.*

*Figure 3.38. Measurement of void underneath Core 2 of Bridge 5624.2O061.*

*Figure 3.39. Measurement of void underneath Core 3 of Bridge 5624.2O061.*


*Figure 3.40. The paving notch and west approach slab of Bridge 5624.2O061, as shown by the borescope.*

***Surveying***

The elevations of the approaches were surveyed and the net elevations were determined following the same procedure as for Bridges 5111.5O034. Figure 3.41 shows the contour maps for the approaches while Figure 3.42 and Figure 3.43 show the depression in contour maps and surface maps, respectively.

The elevation contour maps show the standard drainage elevation across the length of the approach slabs. There appears to be a peak in the pavement after the approach slab in the east approach. For this bridge, the maximum depression was 0.4 inches at the west approach and 0.3 inches at the east approach. There is a 0.5-inch peak at the westbound lane of the west approach. The BI value was calculated as 0.008 for each of the east and west approaches. This value is small compared to values reported by White et al. (2005) which indicates that the bridge approaches are in good condition.

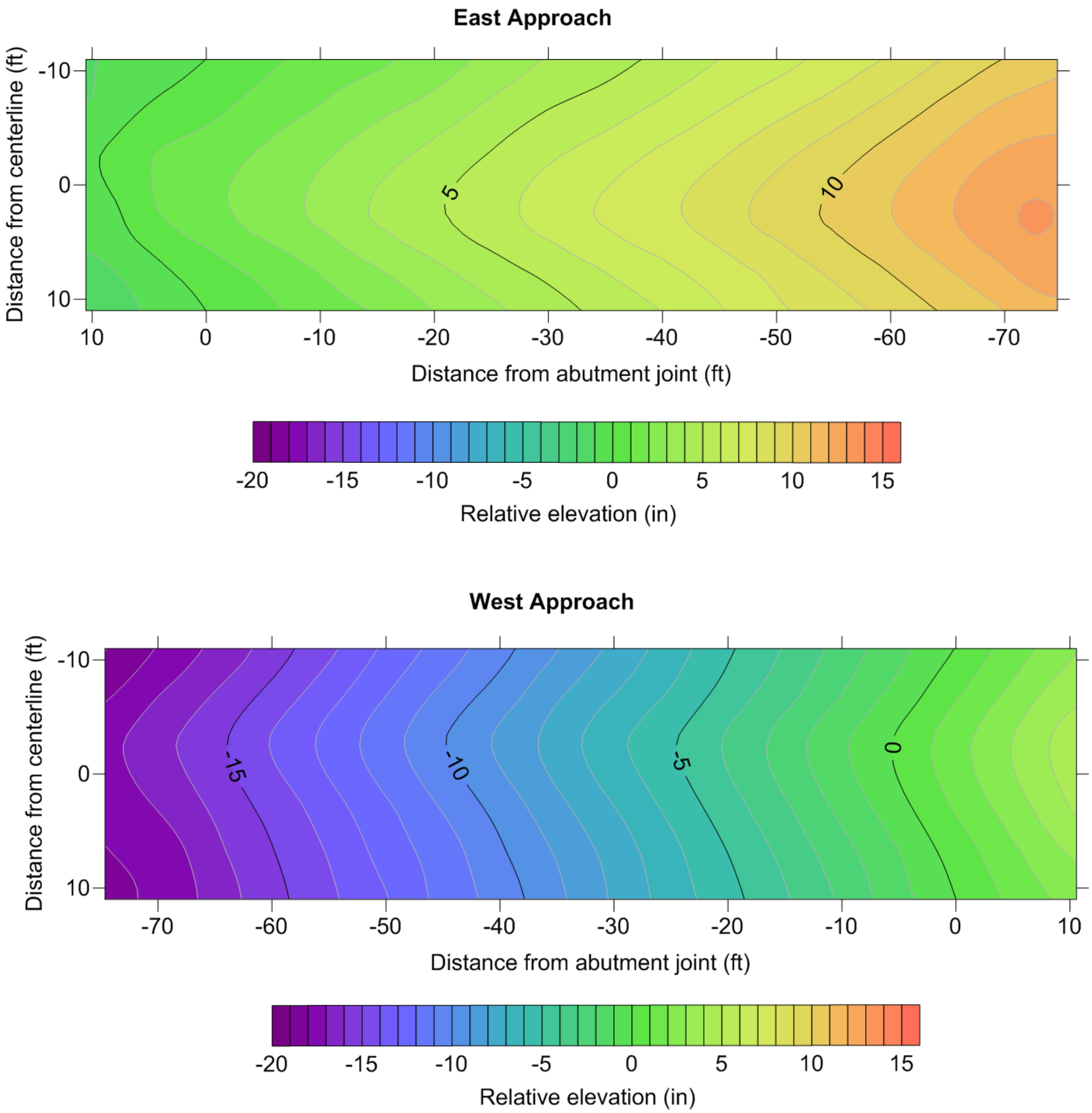



*Figure 3.41. Contour maps of the elevations of the approaches to Bridge 5624.2O061.*

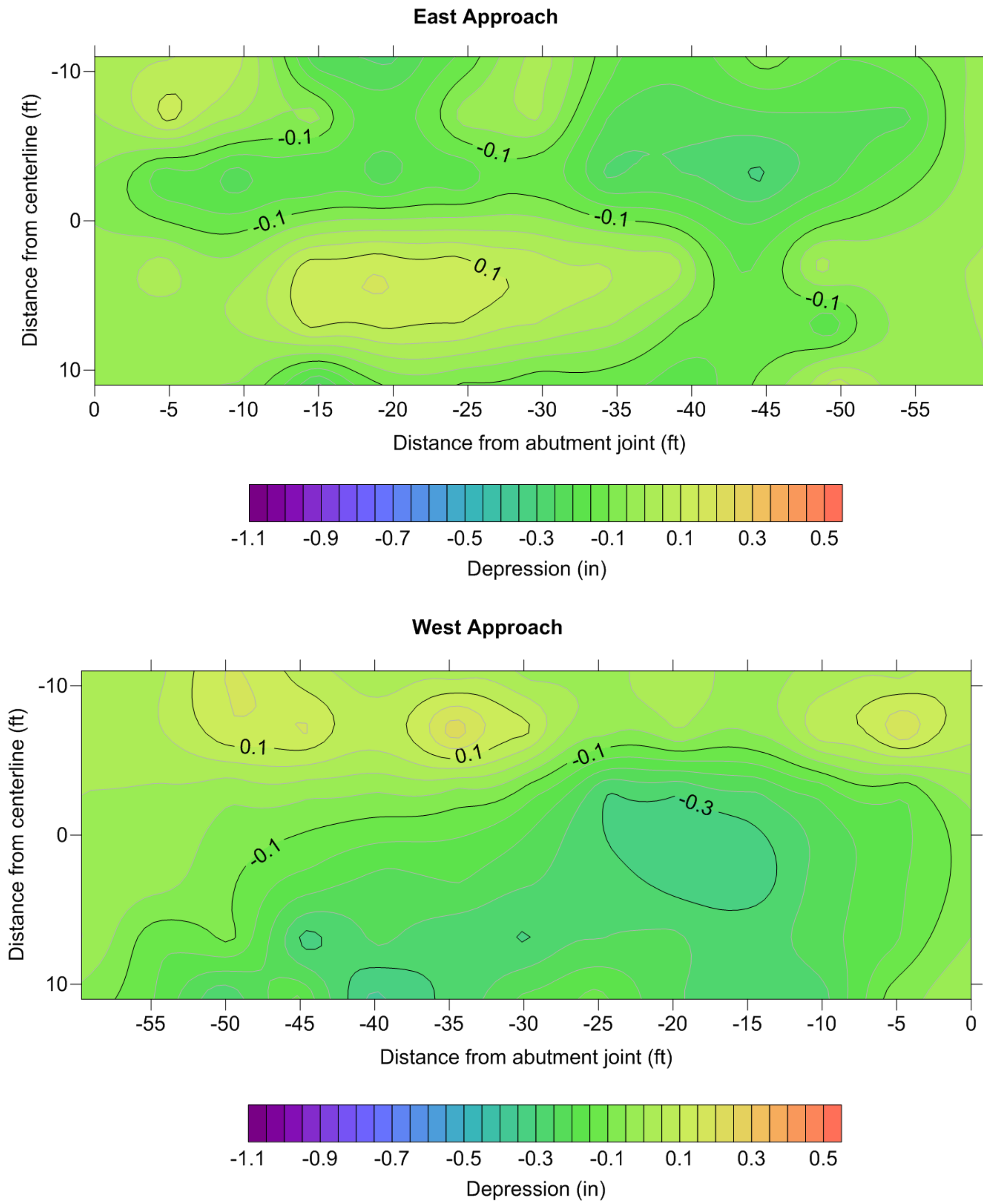



*Figure 3.42. Contour maps of the depression experienced by the approaches to Bridge 5624.2O061.*

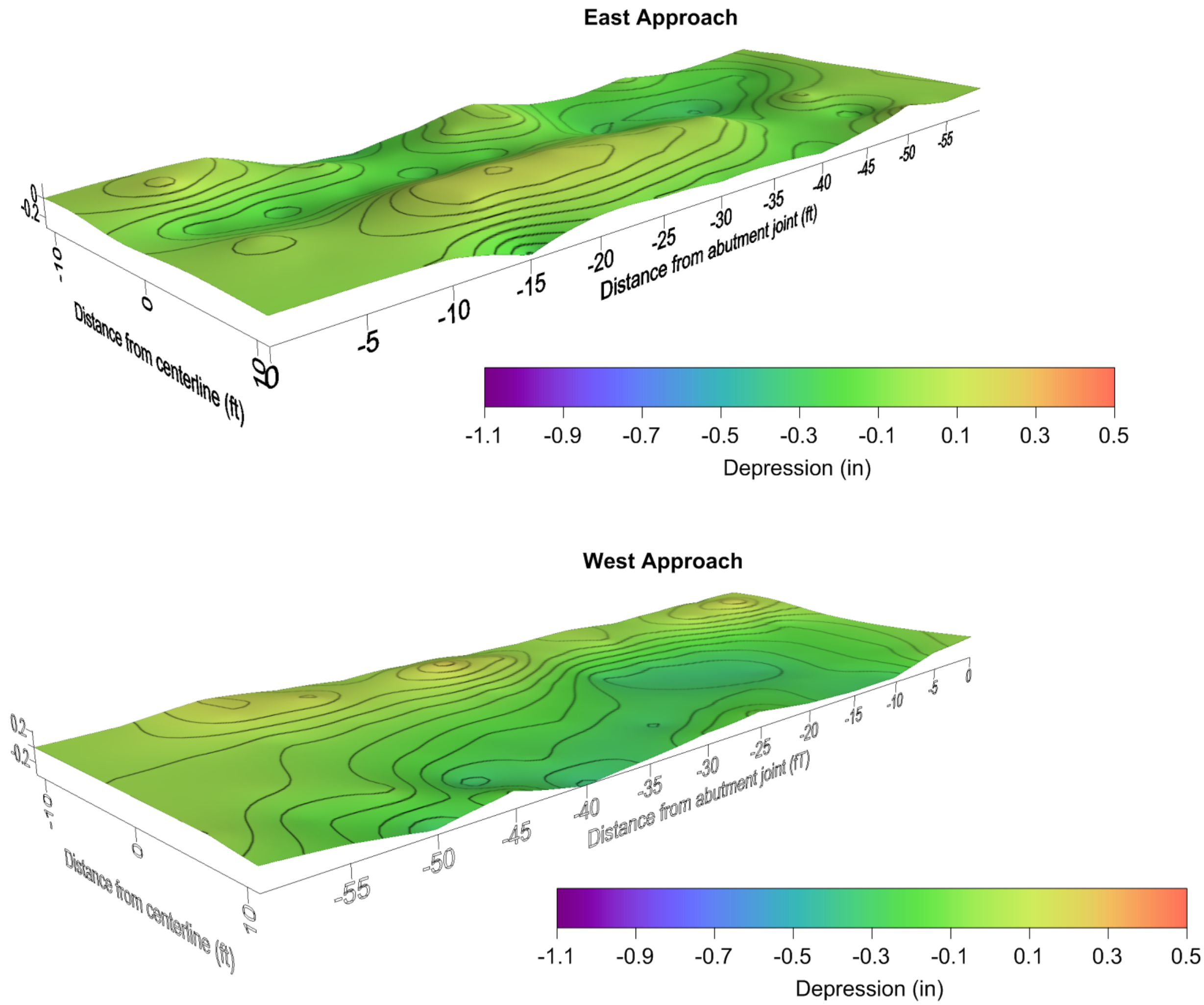



*Figure 3.43. Surface maps of the depression experienced by the approaches to Bridge 5624.2O061.*

### 3.2.5 Bridge 9245.6S001, Route 1 over the Skunk River, Stub Abutments, 2015

Bridge 9245.6S001 is located in Washington County and carries traffic on Route 1 over the Skunk River and an access road. It was built in 2015 with a superstructure consisting of multiple prestressed concrete girders. The bridge has stub abutments and is abutting HMA pavement.

***Visual Observations***

This bridge was in good condition. Only one component showed enough distress to have a rating less than good. However, the slab from the south approach was higher than the bridge, forming a bump. Photographs are provided in Appendix E. Table 3.6 describes the specific features and distresses observed.

*Table 3.6. Details from visual inspection of Bridge 9245.6S001.*

| **Approach** | | **South** | **North** |
|---|---|---|---|
| Pavement surfaces | | Cracks present at the approach area and in the fourth, fifth, and sixth slabs | Cracks present in the fourth, fifth, and sixth slabs |
| Joints | E - Abutment | Filled with debris | Filled with debris at shoulder |
| | First CD | No sealant present | Good condition even though no sealant present<br>Minor unraveling |

***Borescope***
The borescope was inserted into each of the access ports located in the wing walls. Both of the access ports under the north approach were mostly blocked by soil/backfill and both of the access ports under the south approach were fully blocked by soil/backfill. This indicates that either there is no void under the bridge approach or a small void measuring less than the diameter of the camera exists.

### 3.2.6 Bridge 5617.7L061, Route 61 adjacent to 233rd St, Integral Abutments, 2011

Bridge 5617.7L061 is in Lee County and carries traffic on Route 61 over an unnamed road adjacent to 233rd St. It was built in 2011 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***
This bridge was primarily in good condition, but had a few failed joints. The first CD joint in the south approach and both CD joints in the north approach were in poor condition. The west barrier at the south approach was in poor condition as well. The bridge deck and approach slab are level in the lane of the north approach but there is a bump in the shoulder. The EF, or expansion, joints at both approaches produce loud tire noises. Photographs are provided in Appendix F. Table 3.7 describes the specific features and distresses observed.

*Table 3.7. Details from visual inspection of Bridge 5617.7L061.*

| Approach | | South | North |
|---|---|---|---|
| Pavement surfaces | | Spall in pavement at south side | |
| Joints | CF1-Abutment | 1.5-inch width<br>Some sealant squeezing and cracking<br>Some raveling and spalling<br>Vegetation at both shoulders | 1.5-inch width, relatively small compared to other bridges<br>Sealant squeezed or settled and deteriorated some<br>Vegetation in shoulders<br>Build-up of debris at joint with barrier |
| | First CD | 0.5-inch width<br>Scattered portions of sealant still exist, but sealant has failed<br>Some raveling | 0.5-inch width<br>Sealant failed in the lanes<br>Piece of rod exposed |
| | Second CD | 0.5-inch width<br>Many portions of sealant failed<br>Minor raveling | 0.5-inch width<br>Sealant failed in the lanes |
| | EF - Pavement | 6.5-inch width<br>Sealant settled and cracked<br>Some areas are spalled<br>Vegetation is present at shoulders | Filled with depressed rubber<br>3.5-inch width<br>Sealant has settled and cracked but is still present<br>Vegetation present at shoulders and in passing lane<br>Some raveling is occurring |
| | DW or RT | 0.5-inch width<br>Largely intact, but there is evidence of deterioration and spalling | Sealant settled |
| | Contraction | 0.5-inch width<br>Largely intact<br>Joint is inclined | Sealant is in good condition<br>Slab has some cracks at the edge of the lane |
| Shoulder | | Ended after first slab | Ended after fifth slab |
| Barrier | | Curb intact along both barriers<br>No sealant present | Curb is present and there is no void at the bottom of the west barrier |
| Drain outlets | | ----------- | Partially embedded in rip-rap<br>Some vegetation present |

***Borescope***

The borescope was inserted through all four access ports. The west access port under the south approach was mostly blocked with a muddy mixture of fines. The borescope could not reach the approach slab from the east access port under the south approach, so the presence of a void underneath the south approach slab could not be investigated. However, there is an extensive void under the north approach as shown in Figure 3.44. When inserted through the west access port under the north approach, the rebars between the approach slab and the backfill and the back of the abutment wall were visible.


*Figure 3.44. Void under north approach from Bridge 5617.7L061.*

### 3.2.7 Bridge 5657.4O002, Route 2 over Route 61, Integral Abutments, 2011

Bridge 5657.4O002 is in Lee County and carries traffic on Route 2 over Route 61. It was built in 2011 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***

This bridge had condition ratings from good to poor. The only component in poor condition was the barrier on the east approach. The approach slab from the east approach provided a smooth transition to the bridge, but traffic tire noise was loud when crossing the abutment joint. Photographs are provided in Appendix G. Table 3.8 describes the specific features and distresses observed.

*Table 3.8. Details from visual inspection of Bridge 5657.4O002.*

| Approach | | East | West |
|---|---|---|---|
| Joints | Abutment | 2.75 to 3.5 inches in width<br>Heavily deteriorated; sealant is cracked and pieces can be pulled out by hand<br>Raveling and spalling present<br>Vegetation at shoulders and median | 3-inch width<br>Vegetation present |
| | First CD | 0.5-inch width<br>Sealed | Minor raveling |
| | Second CD | 0.5-inch width<br>Sealed | ----------- |
| | EF | 2.75-inch width<br>Sealant has cracked<br>Vegetation present<br>Minimal raveling and spalling<br>Crack present at longitudinal joint | 3.25-inch width<br>Vegetation present |
| | DW or RT | 0.5-inch width<br>Sealant has mostly failed | ----------- |
| | Contraction | 0.5-inch width<br>Sealant has mostly failed | ----------- |
| Shoulder | | Ended after third slab<br>Crack present in shoulder of westbound lane of the first slab | ----------- |
| Barrier | | North: gap is present and erosion has occurred behind the barrier<br>South: joint partially intact | Joints between barriers and deck failed<br>Erosion has occurred behind both barriers |
| Berm slope | | Vegetation present | No erosion under bridge |
| Subdrain outlets | | North drain partially blocked by vegetation | ----------- |

***Borescope***

The borescope was inserted into the south access port of the east approach and both of the access ports under the west approach. The presence of a void under the west approach could not be investigated because both of the access ports were blocked with a mixture of mud, fines, and gravel. The borescope showed that there is a large void underneath the east approach slab; the epoxy-coated rebar, approach slab, and abutment backwall were all clearly visible, as shown in Figure 3.45.


*Figure 3.45. Void under east approach from Bridge 5657.4O002.*

### 3.2.8 Bridge 5627.1O061, J50 over Route 61, Integral Abutments, 2011

Bridge 5627.1O061 is located in Lee County and carries traffic on J50 over Route 61. It was built in 2011 with a superstructure consisting of multiple prestressed concrete girders. The bridge has integral abutments and is abutting PCC pavement.

***Visual Observations***

Most of the joints associated with this bridge were in poor condition while the rest of the components were mostly in good condition. The ride quality was good, although the pavement was a little uneven. Photographs are provided in Appendix F. Table 3.9 describes the specific features and distresses observed.

*Table 3.9. Details from visual inspection of Bridge 5627.1O061.*

| Approach | | East | West |
|---|---|---|---|
| Joints | Abutment | 2-inch width<br>Sealant has cracked<br>Some vegetation and raveling present | 2-inch width<br>Sealant has cracked<br>Some vegetation present |
| | First CD | 0.5-inch width<br>Sealant has failed and is absent in some sections | 0.5-inch width<br>Sealant has failed<br>Spalling and raveling present |
| | Second CD | 0.75-inch width<br>Sealant has failed completely | 0.5-inch width<br>Sealant has failed<br>Spalling and raveling present |
| | EF | 2-inch width<br>Some cracking and raveling | 2.25-inch width<br>Sealant is heavily deteriorated but has not failed yet<br>Vegetation present<br>Cracking and raveling present |
| | DW or RT | No sealant present | 0.25-inch width<br>No sealant present<br>Some raveling |
| | Contraction | 1-inch width<br>Sealant has failed completely | 0.75-inch width<br>One section of sealant has failed |
| Shoulder | | Ended after the first slab | Ended after third slab |
| Barrier | | Seal is missing<br>0.75-inch gap present | North: slab settled; 1-1/8-inch gap<br>South: failure at barrier |
| Subdrain outlets | | ----------- | North: clear<br>South: partially filled with aggregate |

***Borescope***

All four access ports were accessible and were inspected by the borescope. The investigation showed that all the access ports were blocked by gravel and voiding was not noted.

## 3.3 Discussion of Results

Overall, the bridges were in good condition. Table 3.10 provides the condition ratings assigned to each of the bridge components. As can be seen, the pavement surfaces were in adequate to good condition, and the abutment wings, rip-rap slopes, and subdrain outlets were typically in very good condition.

The joints and barriers were in the poorest condition and several currently require maintenance. The sealant in some of the joints was severely deteriorated, while complete failure of sealant was observed in many locations. Spalling and raveling around the joints was noted in several bridges. The majority of the joints had widths that did not agree with the widths provided by the design standard. Slightly smaller widths may be expected because the survey was conducted in the summer. However, the differences between measured and designed joint widths at the abutment joints and the EF joints varied from -2.25 inches to 3 inches. Maintenance of the joints is required at some of the bridges.

*Table 3.10. Overall conditions of inspected bridges.*

| Bridge Conditions | | 5111.5O034 | | 5126.5S078 | | 5622.5O061 | | 5624.2O061 | | 9245.6S001 | | 5617.7L061 | | 5657.4O002 | | 5627.1O061 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Approach | | East | West | East | West | East | West | East | West | South | North | South | North | East | West | East | West |
| Pavement surfaces | | A | G | M | M | G | G | A | A | G | G | G | G | G | G | G | G |
| Joint | Abutment | A | G | A | A | G to P | G | P | P | G | G | A | A | M | M | M | M |
| | First CD | G | G | A | M | P | G to P | P | G | A | G | P | P | A | M | P | P |
| | Second CD | M | G | A | M | G to P | A | P | G to P | G | G | G | P | A | G | P | P |
| | EF or B | G | G | A | A | M | M | G to P | G to P | G | G | G | G | A | M | M | G |
| | DW or RT | A | G | G | G | M | G | G | P | na | na | M | G | G to P | G | P | P |
| | Contraction | A | G | G | G | G to P | P | G to P | P | na | na | G | M | G to P | G | P | P |
| Shoulder | | M | A | M | M | G | G | M | M | G | G | G | G | G | G | G | G |
| Abutment wings | | G | G | G | G | G | G | G | G | G | G | G | G | G | G | G | G |
| Berm slope | | G | G | G | G | G | G | G | G | G | G | G | G | G | G | G | G |
| Subdrain outlets | | na | na | na | na | G | G | G | G | G | G | A | G | A | G | G | A |

*A good condition (G) means that the amount of distress was minor. A fair condition (A) means that the distress was noticeable, but did not require maintenance. A moderate condition (M) means that significant amounts of distress were observed and the component would be expected to require maintenance in the near future. A poor condition (P) means that the component is at the end of its service life. The rating G to P means that the condition of varied from good to poor within the one component.

*Table 3.11. Summary of differences between measured and designed joint widths for abutment joints and EF joints.*

| Bridge | 5111.5O034 | | 5126.5S078 | | 5622.5O061 | | 5624.2O061 | | 9245.6S001 | | 5617.7L061 | | 5657.4O002 | | 5627.1O061 | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Approach | E | W | E | W | E | W | E | W | S | N | S | N | E | W | E | W |
| Joint Type | 'E' Joint | | 'CF-3' Joint | | 'CF-1' Joint | | 'CF-1' Joint | | E' Joint | | 'CF-1' Joint | | 'CF-1' Joint | | 'CF-1' Joint | |
| $W_{des}$* (in) | 1 | | 3 | | 2 | | 2 | | 1 | | 2 | | 2 | | 2 | |
| $W_{meas}$* (in) | 1.75 | 2.0 | 2.5 | 2.5 | 3.5 | 3.5 | 2.12 | 2.75 | NR | 1.25 | 1.5 | 1.5 | 3.5 | 3.0 | 2.0 | 2.0 |
| Δ (in) | 0.75 | 1.0 | -0.5 | -0.5 | 1.5 | 1.5 | 0.12 | 0.75 | n/a | 0.25 | -0.5 | -0.5 | 1.5 | 1.0 | 0 | 0 |
| Joint Type | 'EF' Joint | | | | | | | | | | | | | | | |
| $W_{des}$* (in) | 3.5 | | | | | | | | | | | | | | | |
| $W_{meas}$* (in) | 3.5 | 3.75 | 2.25 | 2.5 | 1.5 | 2.5 | 2 | 1.25 | 3.75 | 4.75 | 6.5 | 3.5 | 2.75 | 3.25 | 2 | 2.25 |
| Δ (in) | 0 | 0.25 | -1.25 | -1.0 | -2.0 | -1.0 | -1.5 | -2.25 | 0.25 | 1.25 | 3.0 | 0 | -0.75 | -0.25 | -1.5 | -1.25 |

*$W_{des}$: Joint width per standard plans; $W_{meas}$: Measured width during field inspections

As indicated in the condition assessment results, large voids were detected underneath most of the approach slabs that were investigated using GPR and/or the borescope. This was confirmed by the collected cores at the four bridges included in the detailed inspections. The only exception was Bridge 5111.5O034 at which very limited voiding was detected. This bridge is 12 years old and was the oldest bridge to be investigated. Also, of the bridges investigated with GPR and/or the borescope, Bridge 5111.5O034 was the only bridge inspected with stub abutments. Integral abutment Bridges 5126.5S078, 5622.5O061, 5624.2O061, and 5657.4O002 had voids that tended to extend further under the approach slab the closer they were to the barriers.

The presence of voids under the integral abutment bridge approach slabs is likely contributed to failure of the joint seal and large gap present between the approach slab and the barrier, a condition that was typically observed in these bridges. As shown in Figure 3.46 through Figure 3.49, the integral bridges had large gaps that would permit directed runoff water to migrate beneath the approach. This would result in erosion of the backfill under the approach slab near the barrier, which would also extend further under the slab as more erosion occurs.

The formation of the gap at the barrier and subsequent erosion of the backfill under the bridge approaches in the integral bridge joints can be attributed to the design philosophy of these systems. In this type of bridges, the bridge superstructure, abutment and wingwalls are integral, which allows for movement of the abutment as the superstructure undergoes length changes due to seasonal temperature changes. The barriers are rigidly attached to the wingwalls and move independent of the approach slabs. WJE often observed remnants of sealant joint material between the barrier and the bridge approach slabs, but the sealant was typically failed and large gaps were present. There appears to have been large differential movement between the barriers and approach slab.

Another reason for the presence of voids beneath the approach slabs of integral bridges is the cycling compression of the backfill adjacent to the abutment and under the approach due to movement of the bridge superstructure and subsequent movement of the abutment during seasonal temperature changes.

Despite of the presence of voids under some of the bridge approaches (with some voids extending approximately 13 feet from the bridge), the bridge approaches generally performed well with little to no distress of the slabs observed in the field. Settlement measurements taken at the joint between the bridge and approach slab as well as the joint between the approach slab and pavement at bridge 5622.O061 and bridge 5624.2O061 are presented in Table 3.12. The measurements reported in the table are an average of at least four differential measurements along the joint length. As can be seen, the largest settlement of 0.4 inches was measured in the west approach slab of bridge 5624.5O061, which also coincides with the largest measured void depth of 6.25 inches. This shows that the slabs can span deep voids without being severely damaged. However with time, continued erosion of the backfill from under the slab may cause the slab to lose its support on soil which may lead to cracking and ultimately failure of the approach.

*Table 3.12. Summary of measured differences in elevation at bridge approach joints*

| Bridge 5622.5O061 - Elevation Difference at Bridge Joint (inches) | | | | | |
|---|---|---|---|---|---|
| West Approach | | | East Approach | | |
| Eastbound | Westbound | Average | Eastbound | Westbound | Average |
| 0.20 | 0.28 | 0.24 | 0.02 | 0.08 | 0.05 |
| Bridge 5622.5O061 - Elevation Difference at Pavement Joint (inches) | | | | | |
| West Approach | | | East Approach | | |
| Eastbound | Westbound | Average | Eastbound | Westbound | Average |
| 0.17 | 0.19 | 0.19 | 0.10 | 0.10 | 0.10 |
| Bridge 5624.2O061 - Elevation Difference at Bridge Joint (inches) | | | | | |
| West Approach | | | East Approach | | |
| Eastbound | Westbound | Average | Eastbound | Westbound | Average |
| 0.35 | 0.46 | 0.41 | 0.02 | 0.20 | 0.11 |
| Bridge 5624.2O061 - Elevation Difference at Pavement Joint (inches) | | | | | |
| West Approach | | | East Approach | | |
| Eastbound | Westbound | Average | Eastbound | Westbound | Average |
| 0.12 | 0.04 | 0.08 | 0.08 | 0.22 | 0.15 |

*Figure 3.46. Gap between north barrier and shoulder at west approach of Bridge 5627.1O061.*


*Figure 3.47. Gap between south barrier and shoulder of west approach of Bridge 5627.1O061.*

*Figure 3.48. Gap between west barrier and shoulder at south approach of Bridge 5617.7L061, note effort to construct a curb along this joint.*

*Figure 3.49. Debris in gap between shoulder and north barrier at west approach of Bridge 5622.5O061.*

# CHAPTER 4. CONCLUSIONS AND RECOMMENDATIONS

## 4.1 Summary and Conclusions

This report provides a summary of results of field inspections of bridge approach slabs and related bridge elements in eight Iowa bridges. The study included a literature review that included the 2005 Iowa DOT study that led to design modifications in approach slabs as well as a review of the design practices from nine other states for approach slabs and abutments. Visual inspection and nondestructive evaluation methods, including GPR testing, elevation surveying, and borescope inspections of abutment access ports, were used to assess the condition of the bridges. The findings of this study can be summarized as follows:

- All of the inspected bridge approaches were generally in a good condition. No excessive settlement or cracking of the approaches were observed, especially at the first slab adjacent to the bridge which is reinforced with two layers of reinforcing. The approach slab and abutment elements, including pavement and shoulder surfaces, rated between moderate and good condition with some cracks visible in some slabs. Minor cracks and raveling were also noted at the joints. Abutment and abutment wings were in good condition in all bridges. Berm slopes generally did not show significant erosion; however, some vegetation was present in some bridges. Sub-drains were generally in good condition; however, some were rated as fair due to partial blockage by vegetation or rocks. Most of the deterioration was observed in the joints and barrier.
- Several of the joints were in moderate or poor condition at which missing or failed sealant was observed. The only exception was bridge 9245.6S001 which was constructed in 2015. Measurements of the joint widths at the bridge and pavement joints indicated variation between the measured and design widths of the joints. Slight variation can be expected in the joint widths; however, excessive variations were noted; in one case the joint width exceeded the design width by 3 inches. Some joints are in current need of maintenance.
- A large gap, measuring more than 1 inch in some cases, was typically observed between the barrier wall and approach slabs of the integral abutment bridges. Deck drainage is often directed at this joint. Remnants of joint sealant material were noted at the gaps; however, all joint seals had failed due to differential movement between the approach slab and the barrier, thereby allowing water draining from the deck to flow through the joint.
- GPR testing indicated the presence of voids under the approach slabs adjacent to the abutment at each of the three integral bridges inspected in detail. Limited voiding conditions were detected at the one stub abutment bridge inspected in detail. The length of the voiding condition in the integral bridges varied. Voids extended approximately 4 feet from the abutment near the centerline and extended further from the abutment at the edges of pavement and barriers. A maximum void length of 13 feet was measured. Cores were collected at all four bridges at which GPR testing was performed to confirm the presence of voids. For the stub bridge, one core was collected which indicated the presence of a shallow 0.25 inch void at the core location. For the integral bridges, multiple cores were collected which confirmed the presence of voids. The depth of voids varied between the core locations, with a maximum void depth of 6.25 inches measured at bridge 5624.2O061.
- Elevation survey data was used to investigate whether settlement of the approach slabs with respect to the bridge and pavement has occurred and to calculate BI values, which is a measurement used to assess bridge approach performance. The results of the survey data showed that the inspected bridge approaches are generally performing adequately. Although some settlement was observed, the maximum settlement measured was approximately 1.0 inch. It is noted that the maximum

settlement typically occurred in the first slab from the bridge towards the midspan of the slab. The maximum differential elevation was measured at bridge 5624.2O061 and was 0.4 inches.

- A borescope was used to inspect the voiding conditions in all the bridges through the access ports installed in the wing walls, except for bridge 5111.5O034 where access ports were not installed. In some locations, the access ports were blocked with debris or soil/backfill which prevented the borescope inspection. The borescope survey at accessible locations enabled the assessment of the voiding conditions under the approach slab, which confirmed the GPR findings. The images from the borescope also indicated presence of voids under two bridge approaches where GPR data was not collected.

Although the inspected bridge approaches are generally performing adequately, the results indicated that several of the joints are in need of maintenance. The results of the inspections showed that large voids extending up to 13 feet in length from the bridge abutment exist under all approach slabs of the three integral bridges included in the detailed inspections while only limited voiding was detected at the inspected stub abutment bridge. The presence of voids under the integral bridges can be attributed to the increased displacement behavior of the abutment but also, at least in part, to the presence of a gap between the bridge barriers and the approach slabs. The failure of the sealant in this gap is likely a result of large deferential movements between the approach slab and the barrier. This gap allows for deck runoff water to erode the backfill below. The saturated backfill likely compacts or erodes more easily due to the cycling compression of the backfill caused by movement of the abutment and bridge superstructure during seasonal temperature changes. Erosion and settlement of the backfill causes the voiding conditions under the approach slabs, which if not prevented, may eventually lead to deterioration and failure of the bridge approach slab due to loss of support.

The difference in the void conditions between the two abutment types is related to their design philosophy. Stub abutments include more components and details and may need more maintenance (compared to integral abutments) as they require an additional expansion joint at the end of the bridge as well as bearings for the bridge girders. Therefore, integral abutments are more widely used by state DOTs in recent years. The results of this study indicate that a disadvantage of the integral abutment design is the formation of voids under the approach slab adjacent to the abutment. A more detailed study of the advantages, disadvantages and life-cycle cost of each abutment type considering all factors, including formation of voids adjacent to the abutment and associated maintenance costs, may be beneficial in determining which abutment type should be used by the state. Modifications to the existing designs may further improve durability and performance.

## 4.2 Recommendations

The recommendations of this study can be summarized as follows:

- Many of the inspected joints had deteriorated or failed sealant. In addition, minor concrete cracking and raveling around the joints was observed. Therefore, it is recommended to develop maintenance plans to improve the condition of the joints. Also, a more frequent inspection and/or maintenance schedule may be required.
- While the presence of access ports did help with the bridge approach inspection efforts, some of the ports were blocked by extraneous material while other ports were completely or partially filled with soil/backfill, even when relatively deep voids were present behind the wall. Therefore, data collected from access ports should be treated with caution. While we were able to clear some of the inspection ports by rodding or drilling, more stringent procedures for sealing the access ports

after original construction and after each use along with a reliable method to clean the access ports prior to each inspection would be useful for future inspections.

- GPR testing was shown to be an effective method for identifying voiding beneath the approach slabs. The procedures used to collect and analyze the GPR data are of critical importance to proper interpretation of the data. Therefore, it is recommended that GPR surveys performed by experienced users be included in future inspections if voiding under the slabs is of primary concern.
- While only one stub abutment was studied, it appeared to have less base erosion and improved performance compared to the integral abutments. However, stub abutments include elements and details that are more difficult to construct and may require additional maintenance. Future studies are needed to confirm this behavior and to assess if the stub abutment design should be preferred to the integral abutment.
- WJE proposes a possible new design detail for a ‘modified stub abutment’ as shown in Figure 4.1. A construction joint could be included at the level of the paving notch in the abutment and the approach slab cast continuous over the abutment. This will eliminate the joint between the approach slab and abutment and will eliminate the paving notch. Further study is recommended to assess the feasibility of this new detail.
- The study indicated that the presence of voids under the approaches of integral bridges can be partially attributed to the presence of a wide joint and gap between the barriers and the slab. Therefore, the following is recommended:
    - Existing bridges: Develop a sealed joint between the approach slab and barrier walls that can tolerate the large differential movement and redirect the flow of water from the bridge away from the joint. The difficulty in sealing the joint is that the primary movement is longitudinal or in-line with the joint gap. Redirecting the primary flow of water off the bridge by installing additional bridge deck drains will also help reduce base erosion resulting from water ingress through the joint.
    - New bridges or rehabilitation of existing bridges: Casting the barrier as part of the approach slab system will eliminate this problematic joint and will result in the barrier moving with the approach slab system. The deck drainage will need to be handled further back from the bridge or drains could be installed within the barrier to contain and carry water away from the approach pavement. This option should reduce base erosion and require less maintenance, but will require a change in the design details currently implemented by Iowa DOT.
- This study did not include wide bridges with significant skews where one shoulder of the approach may have significantly different length than the opposite shoulder. If the standard plans are followed, the reinforcing steel may not be designed to account for the different thermal deformations across the width of the pavement in which case the joint would experience local deterioration. To assess whether the reinforcing steel tie bars are sufficient and compare the robustness of this design to the designs considered in this study, additional investigation of bridges with significant skew angles would need to be conducted.

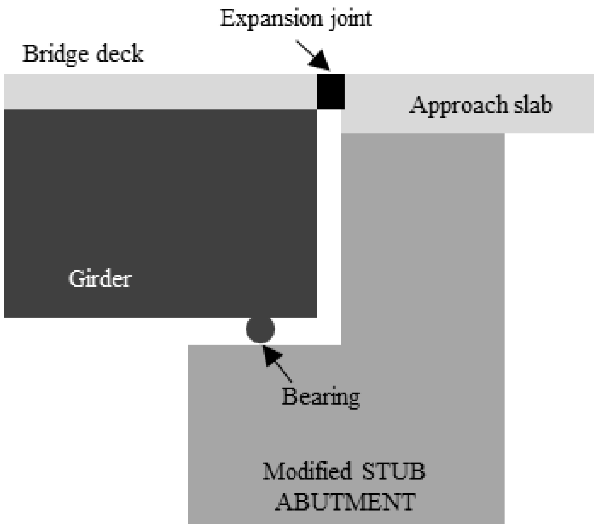



*Figure 4.1. Schematic of the proposed detail for 'modified stub abutment'.*

## 4.3 Implementation

The contents of this report may be used to update or revise the inspection protocol of Iowa DOT for inspection for voiding under the approach slabs. In addition to the installation and use of the borescope access ports, the inclusion of nondestructive evaluation methods, such as GPR, may be implemented in future inspection efforts to identify the extent of voiding beneath approach slab pavements. Also, revisions of the inspection and maintenance intervals for the joints may be considered given the results of this study.

The literature review results for practices of other state DOTs can be used by Iowa DOT to investigate the different approaches used by states. The literature shows that other state DOTs practices are in general agreement with Iowa DOT standard road plans for approach slabs, with exception of the location of the expansion joint. Some states move the expansion joint away from the bridge and place it between the approach slab and roadway pavement.

The findings of this report indicate that some revisions to the design of stub and integral abutments may be beneficial. Modifications to the barrier and approach slab connection is recommended to seal or eliminate the gap that develops between the two elements and leads to water intrusion and erosion of backfill material. The contents and references provided in this report could be used to revise the Standard Road Plans for design of Bridge Approach Pavement (BR).

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