Management of water and backfill characteristics for improved bridge approach performance

by

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A thesis submitted to the graduate faculty
in partial fulfillment of the requirements for the degree of
MASTER OF SCIENCE

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2004

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ABSTRACT

Bridge approach settlement is a problem that draws upon resources for maintenance and repair, causes damage to vehicles, distracts drivers, and creates a negative perception of the transportation agency. In Iowa, void development, which results from soil collapse and erosion, under bridge approach pavement is believed to be a major cause of the resultant problem.

To alleviate bridge approach settlement, erosion and soil collapse must be reduced. This can be accomplished by improving both water management around the bridge and the backfill characteristics.

The purpose of this study was to examine the effects of improved water management and backfill properties on settlement of bridge approach sections and to recommend new alternatives for design, construction and maintenance of new and existing bridge approaches. Furthermore, a threshold limit for bridge approach slab settlement was developed as a criterion to initiate maintenance.

The objectives of this research are (1) Literature review of relevant research and practices of other states, (2) field inspection of existing and under construction bridges, (3) monitoring of maintenance practices in Iowa, (4) laboratory experiments to analyze the properties of various backfill materials and geosynthetic drainage materials, and (5) developing a new rating system to evaluate the performance of bridge approach sections and initiate maintenance.

To analyze the characteristics of backfill materials, the collapse index test was performed to measure the change in volume of the different backfill materials upon saturation at different moisture contents. Vibrating table tests (ASTM D4253-00) were performed on granular backfill materials at several moisture contents to evaluate the minimum and maximum densities, material compactibility, and bulking moisture content

range. A one-fourth scale model of a bridge approach section was constructed in an effort to evaluate geosynthetic drainage materials as well as a variety of drainage details.

This study reveals that poor water management is a major cause of bridge approach problems. Furthermore, granular backfill materials placed within the bulking moisture content range are susceptible to collapse upon saturation and may be a major contribution to the problem. Further, proper compaction of granular backfill materials and placing porous backfill around subdrains is not being followed during construction. Grouting behind bridge abutments and resurfacing of approach slabs, which are the common maintenance practice in parts of Iowa, do not necessarily prevent further settlements. The URETEK Inc. maintenance method, which involves injecting expanding polyurethane beneath the existing approach slab pavement, appears to be a successful alternative to conventional maintenance practices. However, the long term performance of bridge approaches maintained by URETEK Inc. is yet to be verified. The water management bridge approach model illustrates that the worst performance is the drainage model representing the field practices, while using porous backfill, geocomposite vertical drain, and tire chips increase the maximum steady state flow rate and reduce void development and settlement when compared to the Iowa DOT drainage detail. To develop a threshold limit for initiating maintenance practice the Bridge Approach Index parameter was developed. Approach slabs rated less than fair will require maintenance.

The overall technical merit behind this research is the development of improved methods to alleviate the approach slab settlement problems in Iowa. Improving the safety of the bridge structure, driving conditions, vehicle safety, and potentially reducing the maintenance costs are other values of this research.

CHAPTER 1 - INTRODUCTION

1.1 Industry Problem

1.1.1 Problem Statement

Bridge approach settlement is a significant problem that draws upon considerable resources for repair and maintenance at all jurisdiction levels in Iowa. According to the NCHRP Synthesis 234 (1997), bridge approach settlement is a significant problem that affects about 25% of the bridges nationwide at an annual maintenance cost of at least $100 million/year. In Iowa this is equivalent to about 6000 bridges needing repair at an annual cost of about $4 million. The problematic bump contributes to not only added expense and repair time, but added risk to maintenance workers, reduction in transportation agency's public image, distraction to drivers, reduced steering response, damage to vehicles, and in the winter damage to bridge decks from snowplows. Many repair options and alternative design techniques exist, but each has its own drawbacks such as cost, effectiveness, inconvenience to the public, etc. In Iowa, the most common procedure is asphalt resurfacing, which is an ongoing maintenance expense as it addresses the symptom but does not solve the problem.

1.1.2 Goals

The goals of this study were to identify improved design, construction and maintenance practices that will reduce bridge approach settlement problems. In order to recommend improved design, construction and maintenance operations, it was important to understand the processes that lead to the formation of the bump. This was accomplished by documenting the design and practices used in Iowa and other states, and by investigating existing and under construction bridges. Furthermore, a threshold limit for bridge approach settlement was developed to assess the performance of bridge approach slabs and to determine when corrective maintenance measures would be required.

1.2 Technical Problem

1.2.1 Defining Technical Problem

Differential settlement between the bridge deck and the bridge approach can be a result of one or a group of factors such as settlement of the foundation soil, lateral cyclic movement of the bridge structure due to temperature variations, and the difference in settlement rates between the bridge, which is supported on piles, and the approach slab, which is supported on fill material. Therefore, identifying the major factors contributing to bridge approach settlement in Iowa is a very complex process.

From field investigations conducted at 74 existing and under construction bridges, it was concluded that the major reasons for approach slab settlement in Iowa are inadequate water management around the bridge, and the characteristics of the granular backfill used behind bridge abutments.

Evidence of inadequate water management around the bridge was observed at 63% of the bridges inspected. Poor water management can be in the form of poorly sealed expansion joints, blocked or non functioning subdrains, and plugged bridge end drains. Poor water management may also be the reason for other problems to develop around the bridge such as soil erosion at the bridge embankment, along the abutment sides, and under the approach slab.

The characteristics of the granular backfill used behind abutments contribute to bridge approach settlement. The gradation range specified by Iowa DOT for granular backfill includes 20% to 100% passing the No. 8 sieve (2.36 mm). Part of this wide range is included within the range of most erodible soils (Briaud *et al.* 1997). In addition, this material can collapse up to 6% at moisture contents in the range of 3% to 6%, which is the same moisture range measured in the field. Furthermore, Iowa DOT does not require moisture control for placing the backfill material, which if placed at the bulking moisture content range, as observed in the field, can be difficult to compact to the specified density.

1.2.2 Iowa DOT Current Practices to Resolve the Problem

Several maintenance methods have been adopted by the Iowa DOT to resolve the settlement problem. These methods are resurfacing of the approach slab with Hot Mix Asphalt (HMA), grouting under the approach slab, and approach slab removal and replacement. Placing an asphalt overlay at the approach slab is an ongoing maintenance expense as it addresses the symptom but does not correct the problem. Similarly, grouting under the approach slab is not a permanent solution because it does not prevent further settlement from occurring and can restrain movement of the bridge structure. Replacing the approach slab takes place when severe problems such as faulting of approach slab panels develop. Besides being an expensive maintenance option, replacement of the approach slab does not necessarily eliminate bridge approach settlement.

1.3 Research Objectives

The main objectives of this study were to

- Identify improved practices for design, construction, and maintenance of bridge approaches to reduce the bridge approach settlement problem.
- Demonstrate the impact of poor water management and backfill material characteristics on settlement of bridge approach sections.
- Develop practical threshold limits at the interface between the bridge approach and the bridge deck to be used for determination of when corrective measures are required.
- Recommend design, construction and maintenance alternatives specific to Iowa conditions which will eliminate the water management problem and improve the backfill characteristics; and thus, reducing the approach slab settlement.

1.4 Research Significance and Benefit

1.4.1 Scientific Merit

The overall technical merit behind this research is the development of new methods to alleviate the approach slab settlement problems in Iowa. Other technical value of this research is the ability to compare and contrast the effects of various components such as backfill reinforcement, geocomposite vertical drain, and tire chips on the drainage capacity, void development, and differential settlement at the bridge approach. This is accomplished through modeling an approach slab section and evaluating different drainage alternatives. Furthermore, the rating system for bridge approaches is a helpful tool to assess when to initiate repair actions.

1.4.2 Broader Benefits of this research

Improving the safety of the bridge structure and potentially reducing the maintenance cost are other values of this research. Moreover, by alleviating the approach slab settlement, driving conditions, vehicle safety, and transportation agency's public image are all improved.

1.5 Document Organization

1.5.1 Chapter 2. Background

This chapter presents relevant research as well as surveying practices of other states. The background section elaborates on pervious research work conducted to both understand and solve the bridge approach settlement problems. Furthermore, in this section a survey of the practices of other states in related topics such as backfill gradation, backfill compaction, drainage details, approach slab to abutment connection, and joint details is described.

1.5.2 Chapter 3. Research Investigation and Results

This chapter is divided into four main sections. The first section presents and discusses the observations from field investigations conducted in all six districts of Iowa. The purpose of this section is to demonstrate the effects of poor water management and backfill characteristics at existing and under construction bridges on the bridge approach settlement problem. This section also presents and discusses current maintenance practices in Iowa including the use of URETEK Inc. maintenance method.

The second section demonstrates the laboratory testing conducted to evaluate the backfill characteristics by measuring the collapse index. Comparing the erodability of different backfill materials to the range of most erodible soils chart provided by Briaud *et al.* (1997) is also presented in this section.

The third section demonstrates the water management bridge approach model assembled to evaluate the current Iowa DOT drainage detail as well as current practices observed in the field. The model also tested various drainage details such as geocomposite vertical drains, tire chips, and backfill reinforcement are combined to formulate an innovative and an efficient drainage detail.

Finally, this chapter describes the rating system developed to evaluate the performance of bridge approaches and to initiate repairs if necessary. The rating system was based on International Roughness Index data (IRI) and profiles of approach slab sections.

1.5.3 Chapter 4. Summary and Conclusion

This chapter summarizes the key findings of this research. The summary includes observations based on previous research work, practices of other states, and field and lab observations. Conclusions were then made focusing on the research goals and objectives.

1.5.4 Chapter 5. Recommendations

In this chapter recommendations are made addressing all problems related to poor water management and backfill characteristics. The recommendations include new drainage details, as well as modifications to the current Iowa DOT specifications. Furthermore, a pilot study was recommended as a continuation of this study to test the effectiveness of the recommended details in alleviating bridge approach settlement.

CHAPTER 2 - BACKGROUND

2.1 Review of Relevant Research and Literature

Bridge approach settlement and formation of the "bump" at the end of the bridge is a problem that has gained national attention, for which better solutions are vitally needed (Ardani 1987; Arsoy *et al.* 1999; and Briaud *et al.* 1997). According to a survey of 61 different transportation agencies (Laguros *et al.*1990a), bridge approach settlement problems are considered significant in almost 70% of the agencies. A more recent survey by Hoppe (1999) reported that 44% of the state DOTs consider the bridge approach settlement a significant problem (See Figure 2.1). Iowa was listed as having a "moderate" problem (Hoppe 1999). However, Iowa DOT personnel believe that the bridge approach settlement problem in Iowa is more substantial (Brakke 2003).

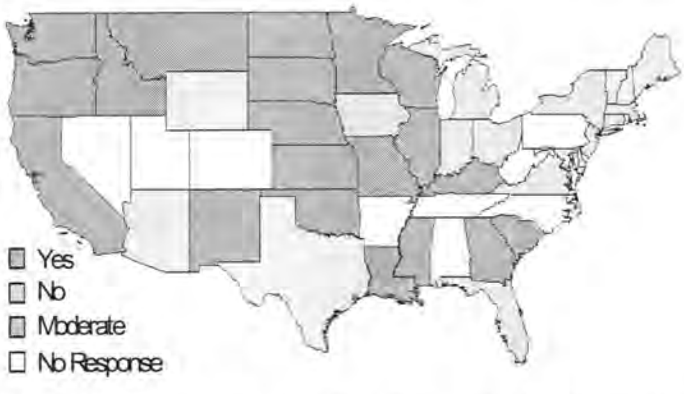


Figure 2.1 - The Significance of Bridge Approach Settlement (Hoppe 1999)

2.2 Review of Bridge Abutments and Approach Slab Design Details

2.2.1 Abutment Details

Bridges are typically classified as integral (movable) or non-integral (conventional or stub) abutment bridges with the main difference between the two types being the connection detail between the bridge superstructure and the abutment (See Figures 2.2 and 2.3).

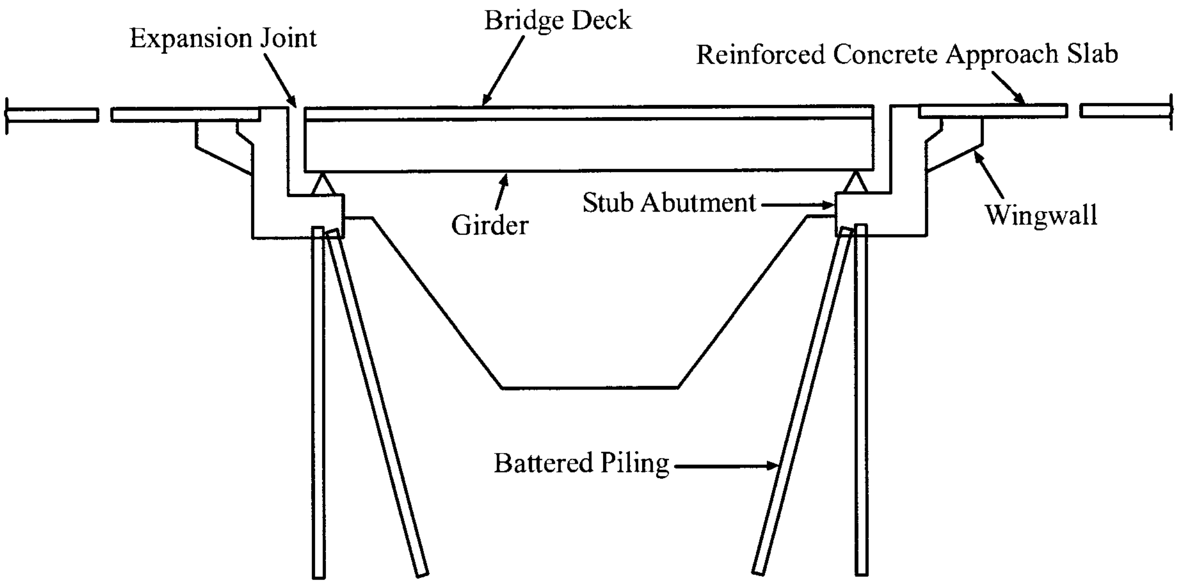


Figure 2.2 - A Simplified Cross Section of a Non-Integral Abutment Bridge (Greimann *et al.* 1987)

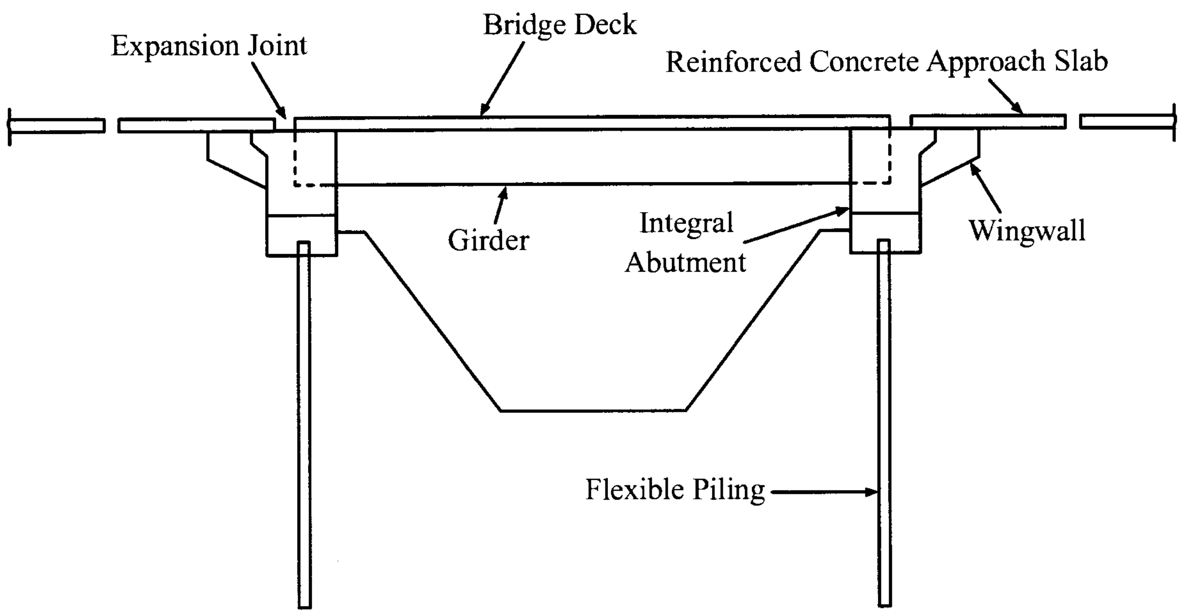


Figure 2.3 - A Simplified Cross Section of an Integral Abutment (Greimann *et al.* 1987)

For non-integral abutment bridges, the superstructure is typically supported on bearing connections that allow for longitudinal movements of the superstructure without transferring lateral loads to the abutment. Battered piles are typically installed to resist lateral soil loads on the abutment backwall. To accommodate the relative movement between the bridge superstructure and the abutment, expansion joints and bearing (slip) connections at each end of the superstructure are typically installed. Increased traffic loads and frequent application of deicing salts during winter, can result in accelerated deterioration of expansion joints and bearing connections, which can lead to costly maintenance problems (Horvath 2000).

To eliminate the use of bearing plates and to reduce potential maintenance problems, a concept was developed to "integrally" or rigidly connect the bridge superstructure to the abutment (Horvath 2000). The use of integral abutments has increased since the 1960's. Integral abutments are usually supported on deep pile foundations using no inclined piles. Since constructing the first integral abutment bridge in Iowa in 1962 (Kunin and Alampalli 2000), their use has increased significantly. In 1997 Briaud *et al.* reported that Iowa had almost 4000 integral abutment bridges.

Greimann *et al.* (1987) and Hoppe and Gomez (1996) reported the following advantages of the integral abutment bridges:

- Simple, and reduced construction and maintenance costs due to the elimination of bearings
- Fewer piles are required for foundation support
- Improved seismic performance

Although, both integral and non-integral bridges are vulnerable to differential settlement, a disadvantage of integral abutment bridges is that they are more affected by the daily temperature changes, which subject the abutment backfill to cyclic lateral loading (Arsoy *et al.* 1999).

Arsoy *et al.* (1999) reported two main problems associated with integral abutment bridges. These problems are:

- Development of a void near the abutment face
- Differential settlement between the bridge superstructure and approach embankment.

Schaefer and Koch (1992) also reported that the lateral movement of integral abutment bridges due to the seasonal expansion and contraction of the bridge superstructure introduce a void near the abutment causing settlement of the approach slab. This cyclic movement also introduces high applied stresses on the pile foundations which may reduce their axial load capacity (Greimann *et al.* 1986). According to Greimann *et al.* (1983) the vertical load carrying capacity of H-piles can be reduced in very stiff clay by 50% for 5.1 cm lateral displacement and 20% for lateral displacement of 2.5 cm.

2.2.2 Approach Slab Details

The approach slab is designed to be supported on the bridge abutment at one end and on the embankment fill or a sleeper slab (or beam) at the other end. The purpose of the approach slab is to minimize effects of differential settlement between the bridge abutment and the embankment fill and to provide a smooth transition between the pavement and the bridge. The performance of the approach slabs depend on many factors including: (1) the approach slab dimensions; (2) the steel reinforcement; (3) the use of a sleeper beam; and (4) the type of connection between the approach slab and the bridge. Hoppe (1999) reported the details of approach slabs used by 39 DOTs (See Table 2.1). Lengths varied from 3 m to 12.2 m and thickness varies 20.3 cm to 45.7 cm.

Table 2.1 - Typical Approach Slab Dimensions Used by Various DOTs (Hoppe 1991)

State	Length (m)	Thickness (cm)	Width limited to	Additional Information
AL	6.1	22.9	Pavement	
AZ	4.6			
CA	3.0-9.1	30.5	Curb-to-Curb	
DE	5.5-9.1			
FL	6.1	30.5	Curb-to-Curb	
GA	6.1-9.1	25.4	Curb-to-Curb	
IA	6.1	25.4-30.5	Pavement	Length varies with skew angle
ID	6.1	30.5		Length varies with skew angle
IL	9.1	38.1	Curb-to-Curb	
IN	6.2			Length varies with skew angle
KS	3.9	25.4	Curb-to-Curb	
KY	7.6		Curb-to-Curb	
LA	12.2	40.6	Curb-to-Curb	Length varies with skew angle
ME	4.6	20.3	Curb-to-Curb	
MA		25.4		Slab is sloped longitudinally
MN	6.1	30.5	Pavement	T-beams
MS	6.1		Curb-to-Curb	
MO	7.6	30.5		Timber header at sleeper slab
NV	7.3	30.5	Curb-to-Curb	
NH	6.1	38.1		
NJ	7.6	45.7		Used with 9.1 m long and 22.9 to 45.7 cm thick transition slab
NM	4.6		Curb-to-Curb	
NY	3.0-7.6	30.5	Curb-to-Curb	Length of Sleeper slab varies with abutment type
ND	6.1	35.6	Curb-to-Curb	
OH	4.6-9.1	30.5-43.2		Length varies with embankment and skew angle
OK	9.1	33.0	Curb-to-Curb	
OR	6.1-9.1	30.5-35.6	Curb-to-Curb	Length varies with fill height and skew angle
SD	6.1	22.9		
TX	6.1	25.4		
VT	6.1			
VA	6.1-8.5	38.1	Pavement	Length varies with skew angle
WA	7.6	33.0	Pavement	Length varies with skew angle
WI	6.2	30.5		
WY	7.6	33.0	Curb-to-Curb	

2.3 Observed Causes that Lead to Formation of the "Bump"

Figure 2.4 shows the possible causes leading to the formation of the bump (Briaud *et al.*, 1997). These causes include: (1) seasonal temperature change; (2) loss of fill material by erosion; (3) poor construction practices (i.e., poor joints, poor drainage, and poor compaction of fill material); (4) settlement of foundation soil; and (5) high traffic loads. However, the two primary causes reported in the literature are the lateral movement of the bridge and the embankment settlement (Schaefer and Koch 1992, Laguros 1990, and Wahls 1990) which are discussed in more details herein.

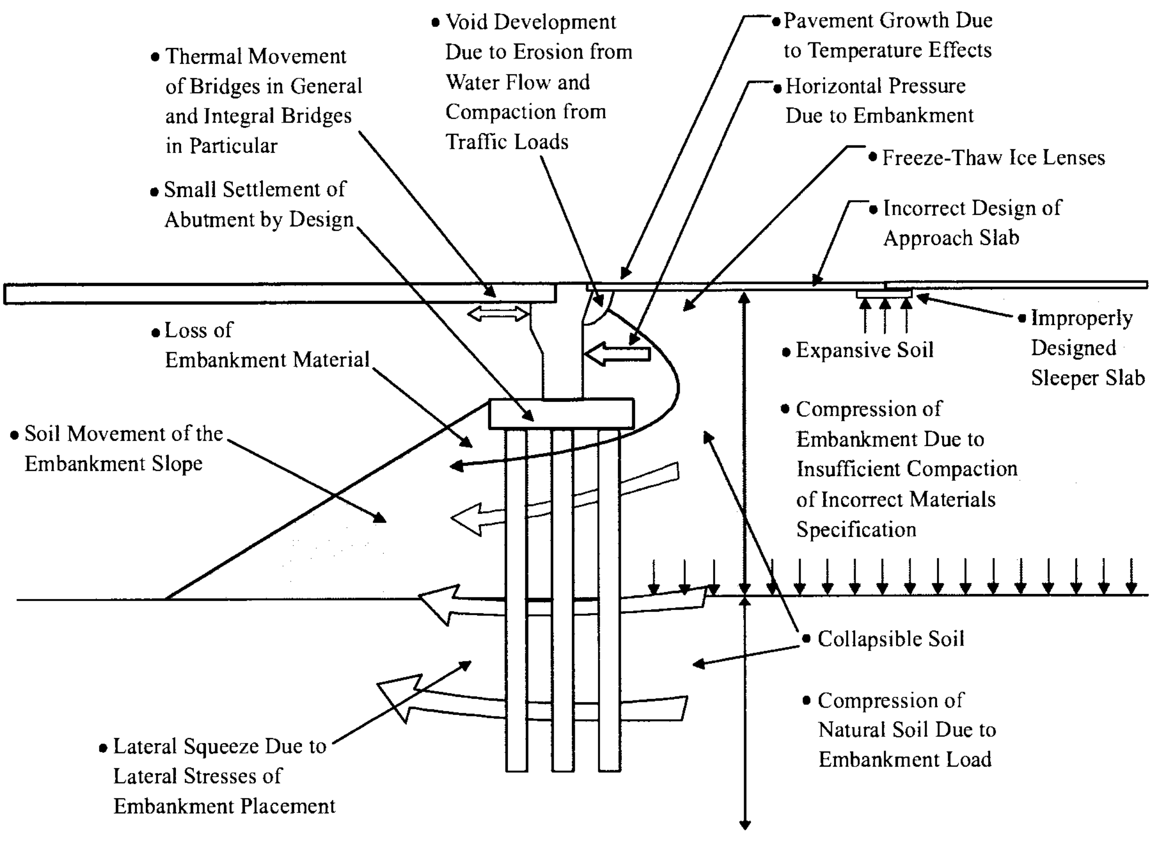


Figure 2.4 - Problems Leading To the Development of the Bump (Briaud *et al.* 1997)

2.3.1 Lateral Movement of the Bridge

Because of seasonal air temperature fluctuation and concrete thermal strain characteristics, the bridge superstructure expands and contracts. For integral abutment bridges, as the temperature changes, the bridge superstructure and the abutment move together which results in subjecting the approach fill and the foundation to cyclic loading. As the temperature increases, the superstructure and the abutment move toward the retained soil causing high lateral stresses which may be greater than the passive pressure limit (Schaefer and Koch 1992). As the temperature decreases, the superstructure and the abutment moves away from the compressed soil leaving a void. As the weather gets colder, the abutments move further away from the retained soil which increases the size of the void between the soil and the abutments (See Figure 2.5). The formation of this void may lead to soil erosion with the presence of water which increases the size of the void behind the abutment and below the approach slab.

For integral abutments, Arsoy *et al.* (1999) measured the ambient temperature and the change of a bridge length in Virginia where the maximum expansion and contraction of the bridge coincided with the maximum and minimum ambient temperatures.

For a bridge of length L subjected to a uniform temperature, the thermal deformation ΔL due to a change in temperature of $\delta T = T - T_o$ is

$$\Delta L = \alpha(\delta T)L \tag{2.1}$$

where α is the coefficient of linear thermal expansion. For concrete α is approximately 6.0 x 10^{-6} per °F (Chen 1995).

Girton *et al.* (1991) idealized the bridge by dividing it into sections with uniform properties using temperature measurements for two Iowa bridges — Hwy 30 Boone River Bridge (concrete girders) and Maple River Bridge (steel girders) located in northwest Iowa. To estimate lateral extension, Equation 2.2 shows that a bridge can be divided into "*n*" segments, where each segment "j" has a uniform coefficient of expansion "a_j", a uniform temperature "T_j", a uniform modulus "E_j" and an area "A_j". Based on the measured temperatures and expansion, the following values were recommended:

- Thermal expansions coefficients of 0.0000045 and 0.000005 in/in/°F, and
- Temperature variation of 150 °F to 140 °F for Boone and Maple bridges, respectively.

$$\Delta = \frac{\sum_{j=1}^{j=n} a_j \Delta T_j E_j A_j}{\sum_{j=1}^{j=n} E_j A_j} L \tag{2.2}$$

The movement of the bridge abutment due to the seasonal temperature also affects the pile stresses and behavior. Girton *et al.* (1991) measured the maximum pile stress which was found to be 60% and 75% of the nominal yield stress at Boone River Bridge and Maple River Bridge, respectively. Lawver *et al.* (2000) reported that the maximum measured pile stresses were slightly above the nominal yield stress of the pile. Greimann *et al.* (1986) performed a three dimensional nonlinear finite element analysis to study the pile stresses and pile soil structure interaction of integral abutment bridges. They concluded that the thermal expansion of the bridge reduces the vertical load carrying capacity of the piles.

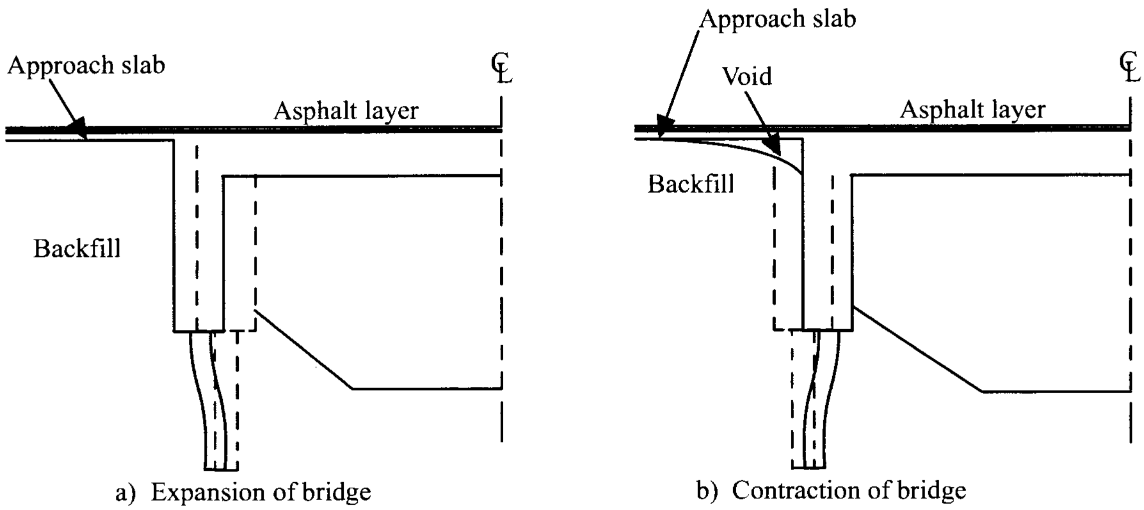


Figure 2.5 - Movement of Bridge Structure with Temperature (Arsoy *et al.* 1999)

2.3.2 Embankment Settlement

In addition to the temperature change effects, embankment settlement is a primary reason causing bridge approach settlement (Wahls 1990 and Holtz 1989). Embankment settlement may be caused by settlement of foundation soil, poor compaction of fill material, poor drainage, and/or loss of fill material by erosion.

Many State Highway agencies investigated the causes of the bump in their states and reported embankment settlement as a primary reason. In Colorado, Ardani (1987) attribute bridge approach settlement problems to: (1) time dependent consolidation of foundation soil and the approach embankment; (2) poor drainage and soil erosion around the abutment; and (3) poor compaction of backfill adjacent to the abutment. Studies in Nebraska (Tadros and Benak 1989) and Kentucky (Hopkins and Scott 1970; Hopkins 1973; and Hopkins 1985) concluded that consolidation of the foundation soil is the primary factor leading to formation of the bump. In California, however; Stewart (1985) reported that the most important factors causing the bridge approach settlement were compression of the embankment fill material and settlement of the foundation material.

2.4 Defining the Bump

The problem of differential settlement at bridge approach sections can be reduced by maintenance. However, to initiate a maintenance action, a threshold differential settlement or a maximum slope needs to be identified. Walkinshaw (1978) suggested that vertical differential settlement greater than 6.4 cm. results in a poor ride quality. Bozozuk (1978) concluded that tolerable settlements are 9.9 cm vertically and 5.1 cm horizontally. Long *et al.* (1988) and Wahls (1990) suggested the use of a relative gradient of 1/200 as a criterion to initiate a remedial action. Furthermore, International Roughness Index (IRI) data were used by Louisiana Transportation Research Center (LTRC) to identify the riding quality of the bridge approach (Das *et al.* 1999). IRI values at the bridge approach of 10 m/km or greater classify the riding quality of the approach slab as poor or very poor.

2.5 Finding a Solution

The differential settlement at the bridge approach can be reduced using several maintenance, and design and construction practices. These practices are summarized below.

2.5.1 Maintenance

When the approach slab settles excessively, the available repair options typically consist of resurfacing, grouting, or replacement. Resurfacing the approach slab with an asphalt layer compensates for the elevation difference between the approach slab and the bridge; however, it does not necessarily prevent the void propagation under the approach slab, which may lead to further settlement. Tadros and Benak (1989) reported that many of the grouted approaches were badly deteriorated because of cracking between the injection holes. Schaefer and Koch (1992) also concluded that grouting the void did not solve the differential settlement problem. Replacement, which is an expensive maintenance alternative, is usually used when faulting of approach slab panels occurs.

Other new maintenance technologies are available including lifting and realigning the approach slab by filling the void under the approach slab. URETEK, Inc. invented the technology of injecting liquid polyurethane into 1.6 cm drilled holes through the concrete pavement to lift, realign, and fill the void under the approach slab. As the polyurethane expands, the voids under the settled slab are filled, and the necessary lifting forces are applied on the approach slab to lift it to its original position. Polyurethane reaches 90% of its full compressive strength within 15 minutes. The amount of rise can be controlled by regulating the rate of injection. A final elevation within 6.35 mm of the proposed elevation can be achieved. It was reported by URETEK, Inc. that this technology is being used by thirty different state DOTs. This method is unaffected by subsurface temperatures between -18 °C and 38 °C. However, this technology is limited in the presence of frozen subgrades.

2.5.2 Design and Construction Alternatives

Maintenance cost of bridge approaches can be significant. Therefore, it is important to identify the causes of the differential settlement and try to minimize or eliminate them during the design and construction processes.

Previous studies (i.e. Briaud 1997, Wahls 1990, Wahls 1983, Edgar *et al.* 1989, and Ardani 1987) suggested solutions for reducing the differential settlement at bridge approaches. These solutions include: (1) improvement of the foundation soil if necessary; (2) the use of well-graded backfill material; (3) reinforcement of the backfill material using geosynthetics; (4) use of abutments supported on shallow foundations; (5) use of elastic, collapsible inclusion or expandable material behind the abutments; (6) installation of appropriate drainage system; (7) use of filter wrap to prevent soil erosion; and (8) constructing approach slabs with an angle from the horizontal, (pre-cambering). These proposed material and design solutions can be grouped into categories of: foundation soil; backfill material; bridge foundation, approach slab and drainage which are discussed further below.

2.5.3 Foundation Soil

The behavior of the foundation soil beneath both the abutment and the embankment fill is one of the most important factors affecting the performance of bridge approaches according to Wahls (1990). Therefore, an adequate subsurface investigation is an essential task that needs to be conducted at the bridge site. For example, soils like clays and silts are more likely to exhibit long term compression and settlement (consolidation) than gravel or sand.

In the case of soft foundation soil, the soil behavior could be modified using preloading, in situ densification, soil removal and replacement and soil reinforcement. Lightweight fill materials could also be used to reduce the load applied on top of soft foundation soil.

Preloading is one of the most commonly used methods to reduce the post construction settlement and soil improvement. The effectiveness of this technique depends on the time available for consolidation to occur. If the available construction time is less than that required for the foundation soil to consolidate, vertical drains can be used to increase the rate of consolidation.

Lin and Wong (1999) studied deep cement mixing technique to improve the strength of soft clay with high moisture content foundation material to reduce the total settlement and the differential settlement at a bridge approach in China. The deep cement mixing columns were designed in a pattern of decreasing in length away from the bridge abutment. Using deep cement mixing increased the unconfined compression strength by 60 times which result in small and gradual decrease in settlement toward the bridge.

2.5.4 Bridge Foundation

Typically integral abutment bridges are supported on pile foundations. As the bridge superstructure moves with temperature change, the abutment piles are subjected to cyclic displacement which results in a maximum pile and abutment stresses at the location where the piles are embedded in the pile cap. Arsoy *et al.* (2002) investigated the performance of H-pipe, and prestressed reinforced concrete piles subjected to cyclic lateral displacements. It was concluded that H-piles loaded on the weak axis are the best type of tested piles to support integral abutment.

Deep foundations are usually used under integral abutment bridges. Although deep foundations settle, they do not allow the bridge to settle the same amount as the approach embankment. This creates differential settlement at the end of the bridge. The use of shallow foundation reduces the stiffness difference between the embankment fill and the integral abutment bridge which reduces the differential settlement. Shallow foundations are 50% to 60% less expensive and require less construction time than deep foundations, (DiMillio, 1982). Grover (1978) compared the behavior of bridges constructed using shallow and pile foundations in Ohio. In 1961, Ohio DOT performed a survey on bridges constructed on spread footings. It was reported that 80% of the abutments experienced more than 6.4 cm of settlement and 10% experienced more than 10.2 cm of settlement. Ohio DOT then changed the bridge design by requiring the use of pile supported bridge abutments which reduced the settlement of the bridge abutments but created the differential settlement problem at the bridge approach. In 1961, 31% of the bridges had differential settlement while in the middle of seventies, after the use of pile foundations, 63% of the bridge experienced differential settlement at their approach slabs. DiMillio (1982) and Wahls (1983) reported that spread footing can be designed and constructed to provide satisfactory performance with a significant cost reduction in comparison with the cost of deep foundation.

2.5.5 Approach Slab

State DOTs have been using flexible pavement, non-reinforced, and reinforced concrete as approach slabs. Dunn *et al.* (1983) compared the performance of all these approach slab types in Wisconsin and reported that 76% of the flexible approaches were rated poor, 56% of the non-reinforced approaches were rated fair, and 93% of the reinforced concrete approaches were rated good.

Anticipated differential settlement at the bridge approach can be accommodated by pre-cambering the approach slab, (Tadros and Benak, 1989). Wong and Small (1994) conducted laboratory tests to investigate the effects of constructing approach slabs with an angle from the horizontal on reducing the bump at the end of the bridge. Horizontal slab provided a rapid change in surface deformation where the bump was obvious, while the sloping slabs with angles of 5° to 10° provided a smoother transition.

2.6 Other States Practices

2.6.1 Approach Slab-Bridge Connection and Joint Width

Two main different approach slab-bridge connection designs have been used by different states. The first is to connect the approach slab reinforcement to the bridge deck by extending the deck longitudinal reinforcement (See Figure 2.6) or to connect the approach slab to the abutment (See Figure 2.7). The second is to have the approach slab resting on top of the bridge abutment (See Figure 2.8). Hoppe (1999) reported that 71% of the state DOTs use a mechanical connection between the approach slab and the bridge in integral abutment bridges (See Table 2.2). Table 2.3 summarizes the approach slab-abutment connection, and joint width details of twelve state DOTs. According to Wolde-Tinsae *et al.* (1987), the joint between the bridge deck and the approach slab should be able to transfer traffic load, prevent surface water from entering, and to allow expansion as necessary to prevent abutment damage. Connecting the approach slab to the bridge deck or abutment helps in transferring traffic load and preventing significant changes of the expansion joint width at the bridge end, which keeps the expansion joint sealed and the joint material unaffected.

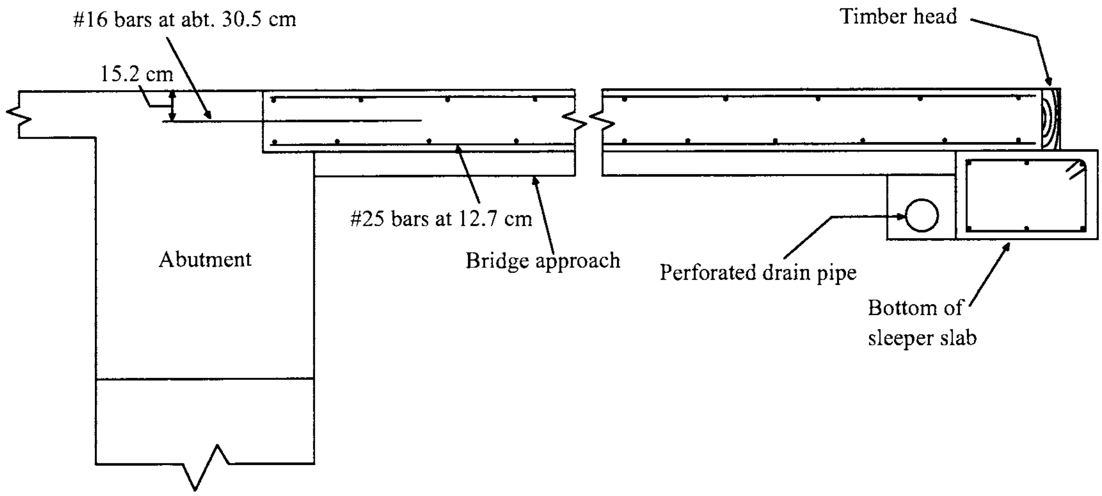


Figure 2.6 - Bridge Approach Connected to Bridge Deck (Missouri DOT 2003)

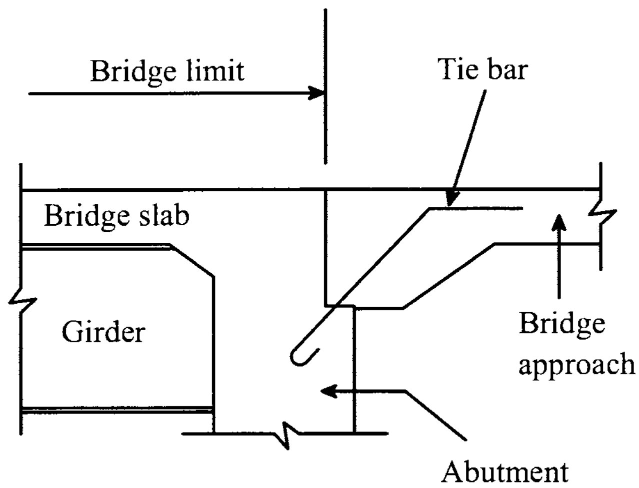


Figure 2.7 - Bridge Approach Connected to Abutment (Ohio DOT 2003)

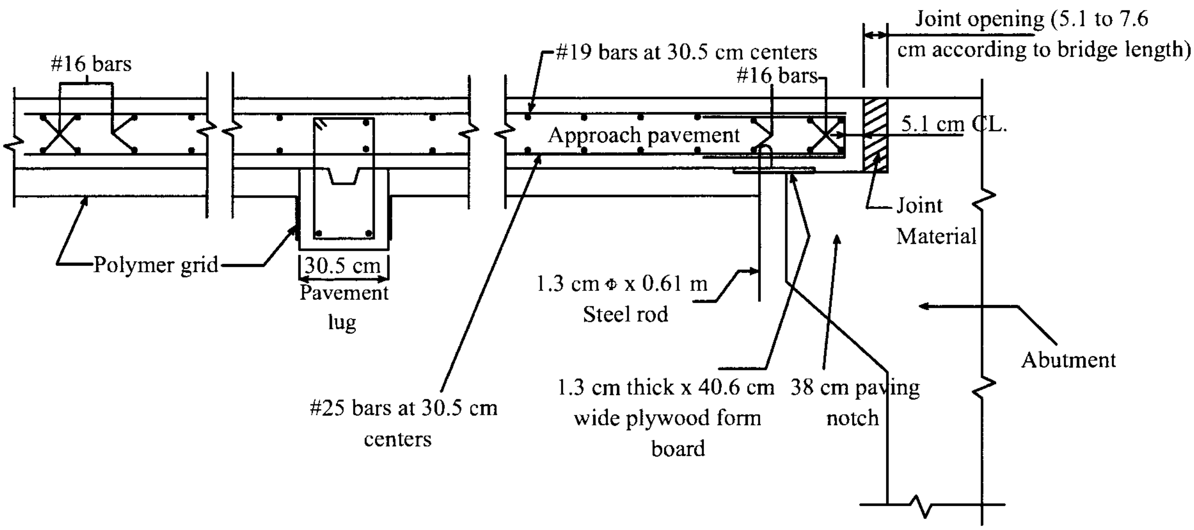


Figure 2.8 - Bridge Approach Resting on Paving Notch (Iowa DOT 2004)

Table 2.2 - Connection between Approach Slab and Bridge (Hoppe 1999)

State	Non-integral Bridges		Integral Bridges		Integral Abutments Not Used
	Doweled or Tied	No Connection	Doweled or Tied	No Connection	
AL	X				X
AZ		X			
CA	X		X		
CT		X			
DE		X			X
FL	X				X
GA		X			
IA	X			X	
ID	X		X		
IL	X		X		
IN		X	X		
KS	X		X		
KY		X			
LA	X				
ME		X	X		
MD					X
MA	X			X	
MN		X	X		
MO	X				
MS		X			X
MT		X			
NV	X			X	
NH	X				
NJ		X			X
ND				X	
OH	X				
OK	X		X		
OR	X		X		
SC	X				
SD		X		X	
TN	X				
TX	X				X
VT	X				
VA		X	X		
WA	X		X		
WI		X			
WY	X		X		

Table 2.3 - Approach Slab-Abutment Connection and Joint Width Details (Hoppe 1999)

State	Connection details	Approach slab to abutment joint width	Sleeper slab
AL	Dowel connected to stub by #19 @ 30.5 cm	*	*
AZ	Vertical dowel	1.3 cm Bituminous joint filler	Yes
FL	Vertical dowel	1.3 cm Thick Expanded polystyrene	*
IA	Inclined dowel @ 30.5 cm centers	2.5 cm Joint opening filled with expansion	No
MO	Horizontal #16 bars @ 30.5 cm bent vertically into abutment (L-type)	No joint	Yes
NC	No connection	5.1 cm Solid opening for joint seal	No
NV	Horizontal slab restrainer @ 60.9 cm	1.3 cm Thick expanded polystyrene	Yes
NY	No connection	Only construction joint	Yes
OH	Diagonally tied to abutment	2.5 cm Performed expansion joint filler	*
OR	#16 x 1.1 m dowels with Std. 180° hook one end @ 30.5 cm	1.9 cm Performed expansion joint filler	Yes
TN	Diagonal #19 @ 30.5 cm into stub and extending horizontally into the abutment wall	1.3 cm V-groove	Yes
WA	L-type anchor; #13 @ 30.5 cm centers	1.3 cm Thick premolded joint filler	No

* - Data not available

2.6.2 Backfill Material

Ideal properties for backfill include: easily compacted, no time dependent properties (e.g. consolidation), resistance to erosion, and elastic. Granular materials are more elastic in behavior than silts and clays, which reduces the non-recovered backfill movement when the abutments move away from the retained soil. Backfill materials used in bridge approach construction are usually selected granular material with some fines. According to Christopher *et al.* (1988), FHWA (2000) recommended the use of a backfill material with less than 15% passing the No. 200 (75 μm). Wahls (1990) recommended the use of materials with a plasticity index (PI) less than 15%, percent, fines less than 5%, and density ranging from 95% of AASHTO T-99 to 100% of AASHTO T-180. Wahls (1990) stated that well graded backfill materials with less than 5% passing the No. 200 sieve are easy to compact with small vibratory compactors, which minimize after construction compression of the backfill and eliminate frost heave problems. CalTrans specified a PI less than or equal 15% and a relative compaction of 95% or more (Christopher *et al.* 1988). Hoppe (1999) reported that 59% of the DOTs responded to his survey use more stringent material specifications for the bridge approach fills with a typical requirement of limiting the percent of soil particles passing the No. 200 sieve between 4% and 20% with the fill placed and compacted in lifts of 150 mm (6 in.) to 200 mm (8 in.). However, 50% of these DOTs had difficulty obtaining the specified degree of compaction in the proximity of the bridge abutment because of compaction equipment space limitation.

Tables 2.4 and 2.5 summarize the backfill gradation required by Iowa and other state DOTs. Most states specify a range of approximately 0% to 10% passing the No. 200 sieve which coincides with Iowa DOT current requirement. Iowa DOT requires that the backfill material shall be deposited in layers not exceeding 200 mm (8 in.) in loose thickness. The first layer shall be compacted to not less than 90% of maximum dry density and each succeeding layer not less than 95% of maximum dry density, which is determined in accordance with Iowa DOT Materials Laboratory Test Method 103. The majority of the states require AASHTO T-99 as a compaction method which is very similar to Laboratory Test Method 103 specified by Iowa DOT (See Table 2.6). However, no moisture content restrictions were specified by Iowa DOT.

Many state DOTs recommended the use of a reinforced embankment behind the bridge abutment. Monley *et al.* (1993) reported that Wyoming Highway Department used a multiple layer of geosynthetic reinforcement within compacted granular embankments since 1983. Edgar *et al.* (1988) reported that none of ninety approach slabs constructed or retrofitted using the geosynthetic reinforced embankment in Wyoming required maintenance or repair after 5 years. However, Wahls (1990) and Horvath (1991) argue that geosynthetic reinforced backfill should be used with a compressible material between the abutment and the backfill to allow for large recoverable cyclic movement. Wahls (1990) stated that this compressible material should provide adequate drainage without soil fines erosion. Horvath (2000) reported two design alternatives including geofoam as a compressible material (See Figure 2.9).

When using geotextiles as backfill reinforcement, Edgar *et al.* (1988 and 1989) reported the use of a collapsible material between the abutment and the backfill material. This material is rigid when dry and collapses to create a void when wetted allowing for the mobilization of tension in geotextile reinforcement. The use of collapsible material reduces the settlement of the embankment fill and the lateral forces on the bridge abutment (Edgar 1989; and Abu-Hijleh *et al.* 2000). Using finite element analysis, Monley *et al.* (1993) concluded that the placement of geotextile reinforcement without a collapsible material does not reduce the approach slab settlement. Many state DOTs have successfully used compressible and collapsible materials behind the abutment. For example North Dakota used a 10.2 cm vertical strip of compressible material and Illinois DOT used non-compacted porous granular material (Wahls 1990; and Kunin and Alampalli 2000). Furthermore, Oregon and Wyoming DOTs used geosynthetic-reinforced embankments with a gap at the bridge wall.

Table 2.4 - Backfill Gradation of Different DOTs

State	Percentage passing						
	Max. sieve size (mm)			4.75 mm (#4)		0.075 mm (#200)	
		min	max	min	max	min	max
Illinois	75	100	100	50	100	0	4
Indiana	50	90	100	20	70	0	8
Kansas	101	100	100	0	60	0	5
Michigan	25	60	100	-	-	0	7
Minnesota	50	100	100	0	50	0	4
Missouri	50	100	100	0	5	-	-
Montana	50	100	100	20	40	0	8
Nebraska	9.5	100	100	92	100	0	3
North Dakota	75	100	100	35	85	0	15
Ohio	75	100	100	-	-	0	20
South Dakota	37.5	100	100	0	20	-	-
Wisconsin	75	85	100	25	100	0	8
Virginia	75	100	100	16	30	4	14
Colorado	50	100	100	30	100	5	20
Washington	50	75	100	22	66	0	5
New York	101	100	100	0	70	0	15
Tennessee	50	100	100	35	55	4	15
South Carolina	50	100	100	30	50	0	12
Oklahoma	75	100	100	0	45	0	10
Kentucky	101	100	100	0	30	0	5
North Carolina	9.5	100	100	80	100	0	20
California	75	100	100	35	100	-	-
Idaho	75	100	100	55	100	0	5
Massachusetts	12.5	55	85	40	75	0	10
Louisiana	12.5	100	100	-	-	0	10
Nevada	75	100	100	35	100	0	12

Table 2.5 - Iowa DOT Backfill Gradation

Sieve size (mm)	Sieve no.	% passing
76.2	3"	100
2.36	# 8	20-100
0.075	# 200	0-10

Table 2.6 - Compaction Requirements for Various States

State	% of dry density	Method
Illinois	95	AASHTO T-99 C
Indiana	95	AASHTO T-99
Kansas	95	AASHTO T-99 C/D
Minnesota	95	AASHTO T-99
Missouri	95	AASHTO T-99 C
Nebraska	100	AASHTO T-99
North Dakota	95	AASHTO T-99
Ohio	102	AASHTO T-99
South Dakota	95	AASHTO T-99
Wisconsin	95	AASHTO T-99 C
Colorado	95	AASHTO T-180
Washington	95	AASHTO T-99
New York	95	Standard Proctor
Tennessee	95	AASHTO T-99 C
South Carolina	95	AASHTO T-99 A/C
North Carolina	95	AASHTO T-99
California	95	Standard Proctor
Idaho	95	AASHTO T-99 A/C
Massachusetts	95	AASHTO T-99 C

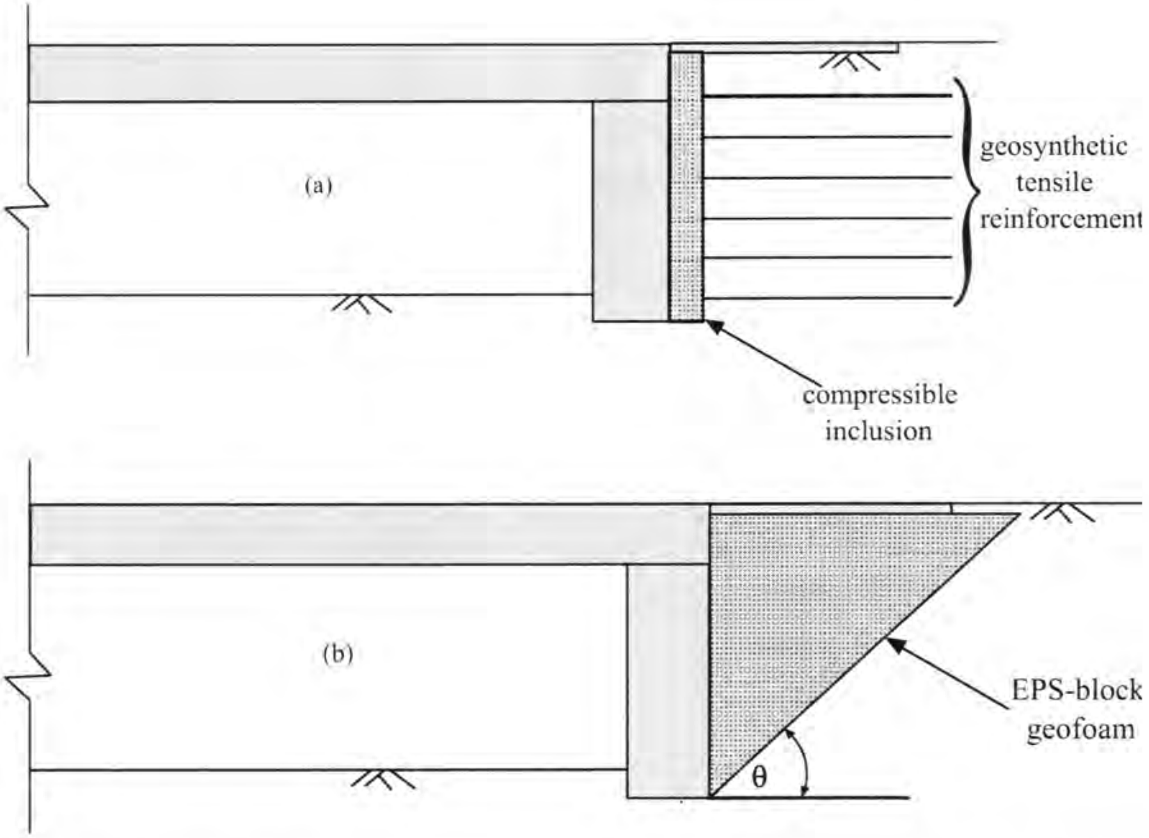
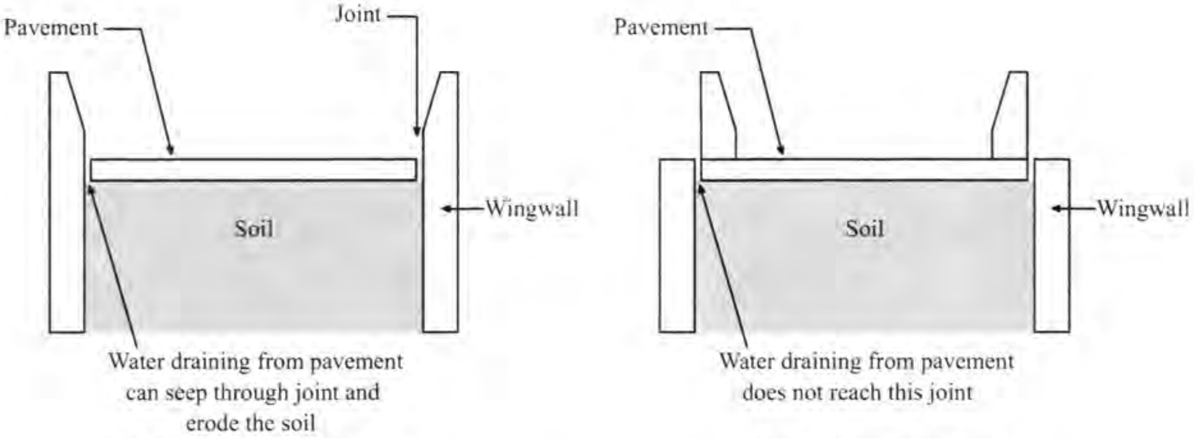


Figure 2.9 - Two Design Alternatives to Alleviate the Differential Settlement Problem Using Geosynthetic-Reinforced Backfill and Geofoam (Horvath 2000)

2.6.3 Drainage

Water that is collected on the bridge surface can cause significant damage to the approach. Water that flows down between the abutment and the bridge approach through joints or cracks or flow around the bridge can erode the backfill if not drained properly. Therefore, an effective method is necessary to drain rainfall runoff. According to Briaud *et al.* (1997) both surface and subsurface drainage need to be considered. Surface runoff should be directed away from the bridge joints, which could be achieved by constructing the wingwall as shown in Figure 2.10.

A survey of the different drainage designs implemented by other states DOTs was carried out in order to evaluate the current Iowa DOT design as well as decide on the optimum drainage system suitable for bridge approaches in Iowa. The results of the survey showed that there are three main categories of drainage systems: (1) using porous backfill around the perforated drain pipe; (2) wrapping geotextiles around the porous fill; and (3) using vertical geocomposite drainage system (See Figures 2.11 to 2.13). Wrapping the porous fill with geotextiles keeps it from eroding and preventing the drainage pipe from getting plugged. Some states combined two or more of these categories together to increase the drainage efficiency (See Table 2.7). This table show that out of 16 states only 2 use porous fill wrapped with geotextile in combination with vertical geocomposite drainage.



(a) Not recommended **(b) Recommended**

Figure 2.10 - Recommended Wingwall Detail (Briaud *et al.* 1997)

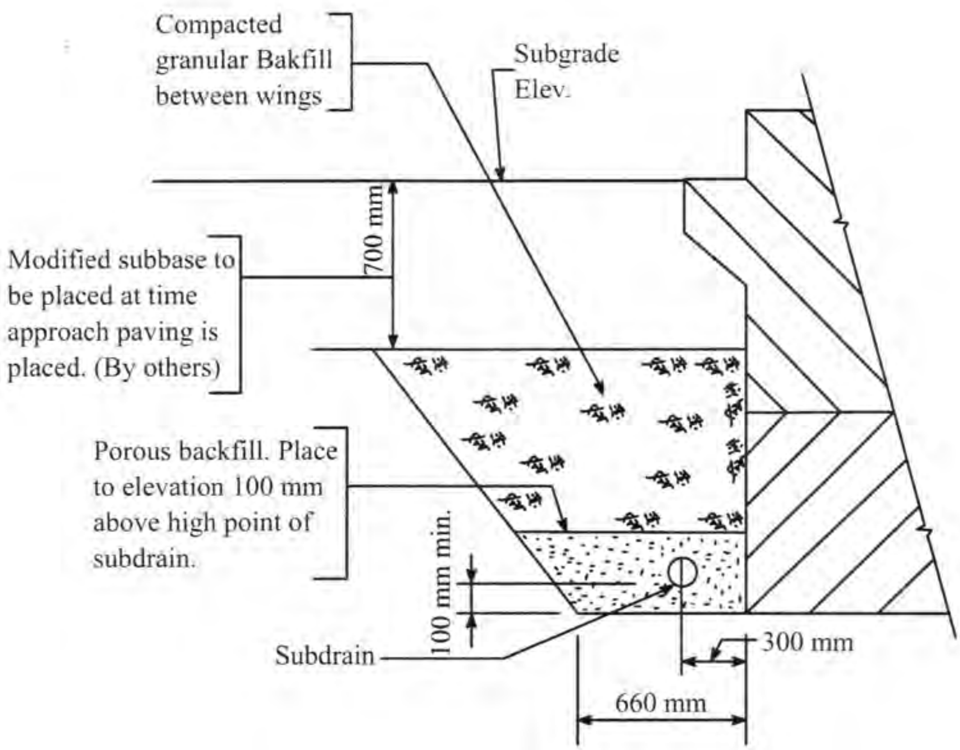


Figure 2.11 - Porous Fill Surrounding Subdrain (Iowa DOT 2004)

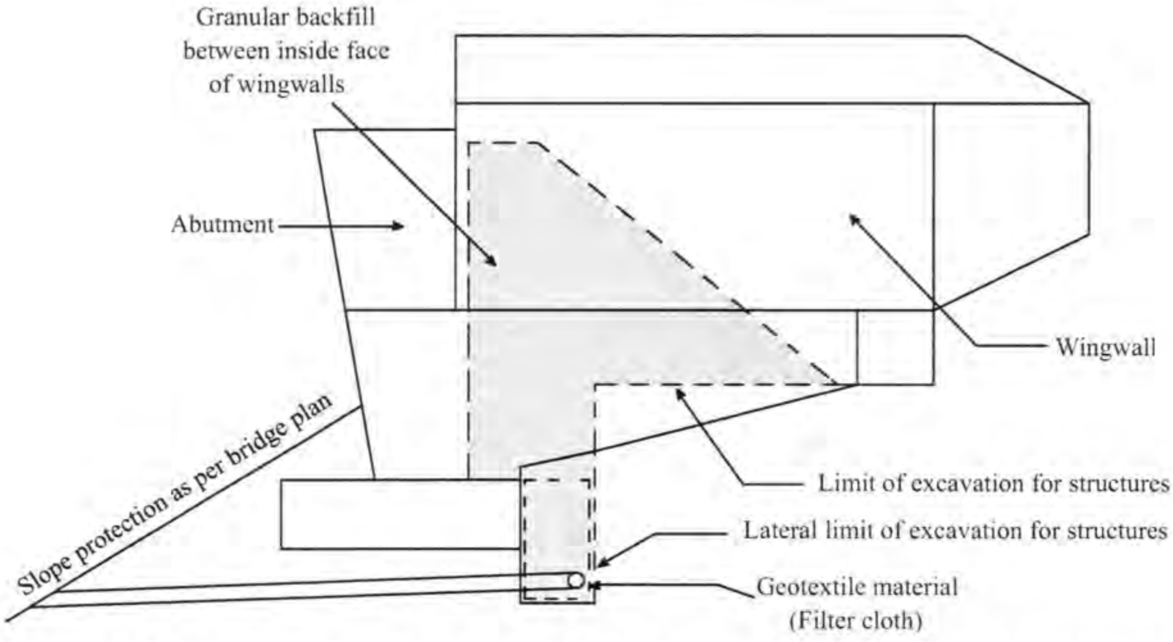


Figure 2.12 - Granular Backfill Wrapped with Geotextile Filter Material (Wisconsin DOT 2003)

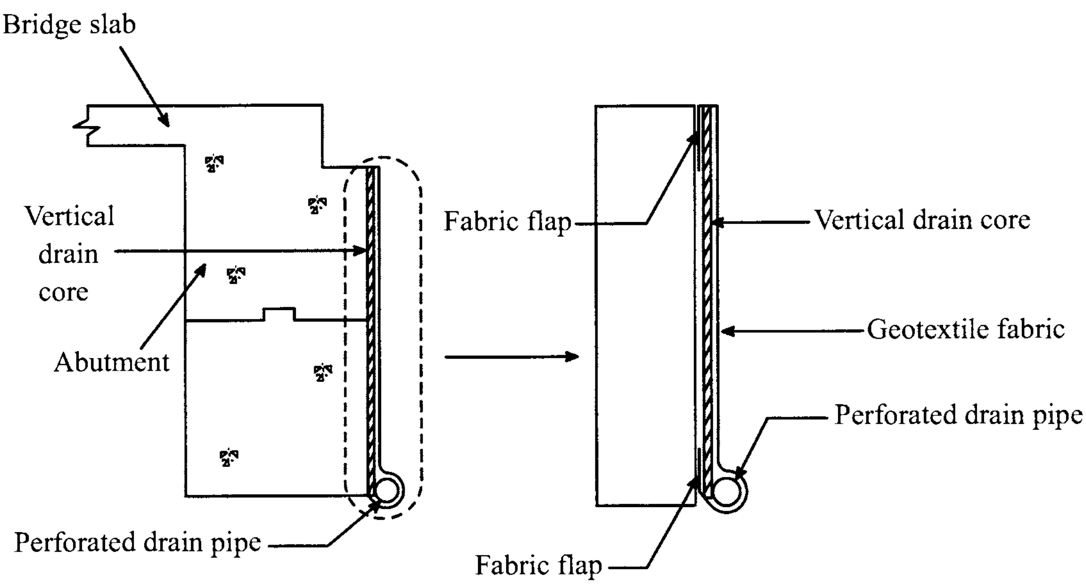


Figure 2.13 - Geocomposite Vertical Drain Wrapped with Filter Fabric (Missouri DOT)

Table 2.7 - Drainage Methods Used by Various States

State	Porous fill	Geotextile	Geocomposite drainage system
Iowa	x	-	-
California	x	x	x
Colorado	-	x	x
Indiana	x	x	-
Louisiana	x	x	x
Missouri	-	x	x
Nebraska	-	x	x
New Jersey	x	x	-
New York	-	-	x
North Carolina	x	x	-
Oklahoma	x	x	-
Oregon	x	x	-
Tennessee	x	x	-
Texas	x	x	-
Washington	x	-	-
Wisconsin	x	x	-

CHAPTER 3 - RESEARCH INVESTIGATION AND RESULTS

3.1 Introduction

From the previous researchers' observations, it was concluded that there are many factors that contribute to bridge approach settlement; however, not all the factors mentioned by previous researchers contribute to bridge approach settlement in Iowa. Therefore, field investigations at 74 bridges in all six districts of Iowa were performed to identify the major factors responsible for the settlement of bridge approaches as well as to estimate the severity of the problem.

From the field observations it was concluded that water management around the bridge and the characteristics of the granular backfill material used behind bridge abutments are major factors associated with bridge approach settlement in Iowa. As a result, the collapse index test was developed to characterize the behavior of backfill materials, and the water management bridge approach model was assembled to evaluate current Iowa DOT drainage designs, and alternative drainage details that can improve the water management around the bridge.

Finally, bridge approach profiles at U.S. 65, International Roughness Index (IRI) data, and Iowa DOT ratings were combined and used to develop a rating system for bridge approaches to indicate when corrective measures are required.

3.2 Field Investigations

Seventy four existing and under construction bridges were investigated. The locations of these bridges are shown in Figure 3.1. Twenty two bridges in district 1, fourteen bridges in district 2, twelve bridges in district 3, eleven bridges in district 4, seven brides in district 5, and eight bridges in district 6 were investigated. Figure 3.2 shows a bridge approach section with no problems, while Figure 3.3 summarizes the frequent problems observed at the investigated bridges.

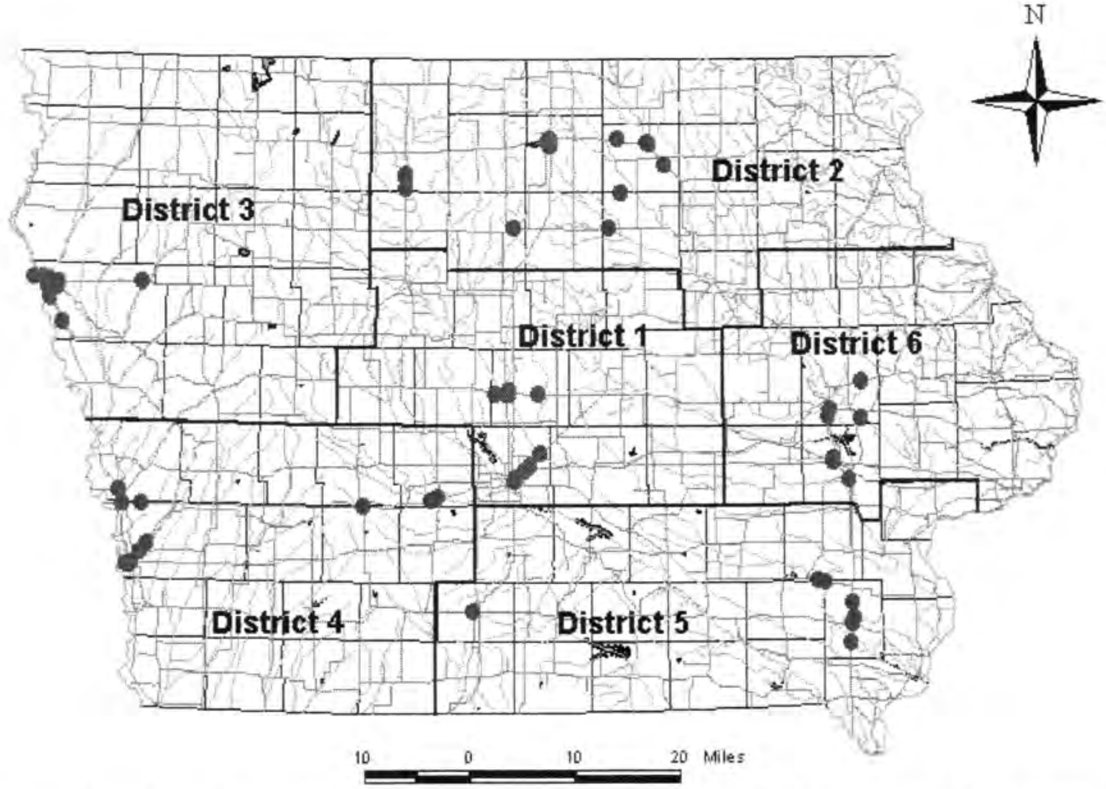


Figure 3.1 - Iowa Map with the Location of the Inspected Bridges at All Iowa Districts

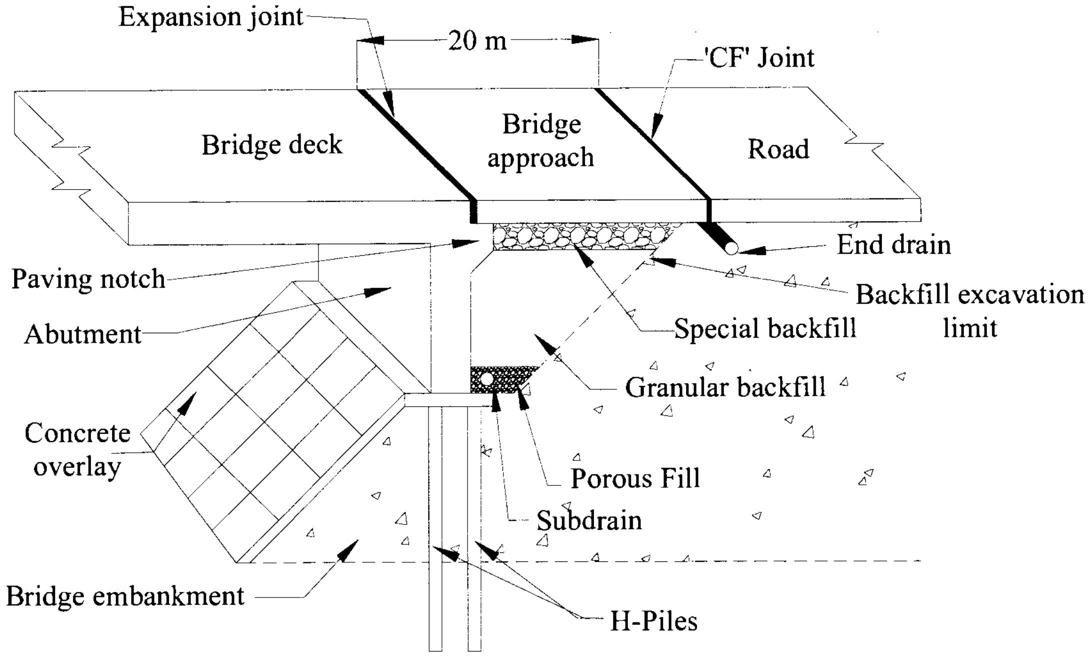


Figure 3.2 - Schematic of a Bridge Approach Section with No Problems

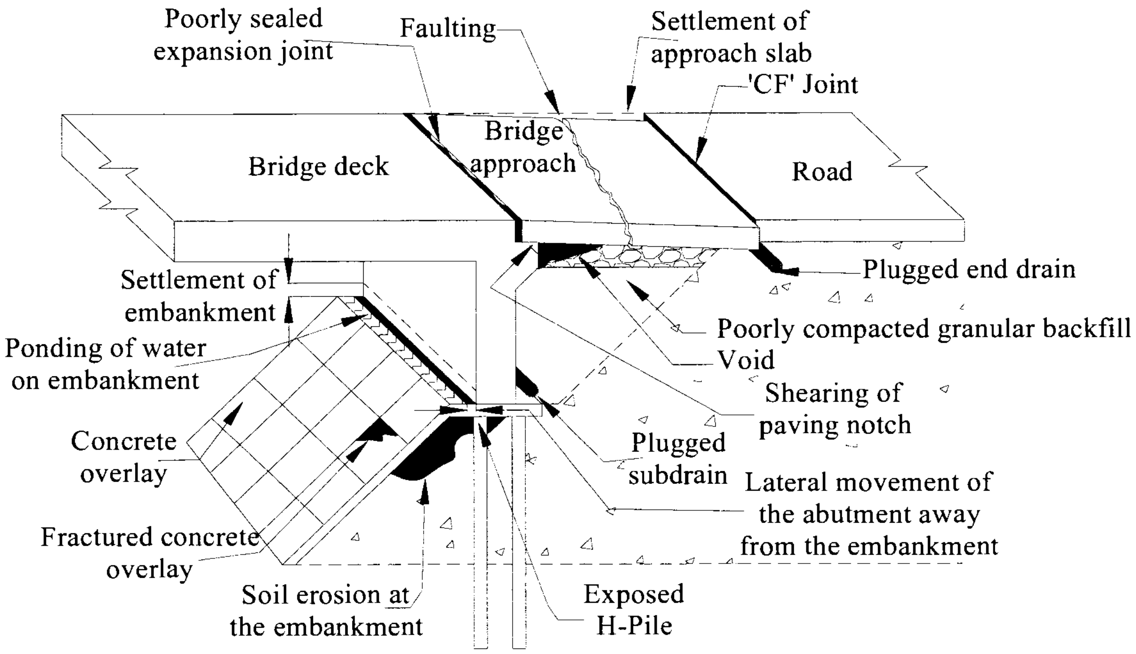


Figure 3.3 - Schematic Summarizing the Frequent Problems Observed at the Investigated Bridges

3.2.1 Existing Bridge Approach Sections

Sixty six existing bridges were investigated. The main factor observed contributing to bridge approach settlement at existing bridges was poor water management around the bridge since it was observed at 63% of the inspected bridges. Table 3.1 presents a summary of all the problems observed during the field inspections. In this section however, bridges with evidence of poor water management are presented in details.

Table 3.1 - Summary of Problems Observed at Bridges with Poor Performance

District	Bridge no.	Construction date	Structure	Crossing over	Major problems
1	U.S. 65 over South Skunk River	-	Integral abutment	South Skunk River	• Differential settlement • Lateral movement of the abutment
1	*U.S. 65 over Rail Road (Mile 89)	-	Non Integral abutments	U.P. Rail Road	• Expansion joint not sealed • Damage to expansion joint • Soil erosion at all four ends of the bridge • Differential settlement
1	*7783.1L065	1993	Integral abutments	U.S. 6	• Settlement of the bridge embankment • Differential settlement
1	U.S. 65 near mile 78	-	Non integral abutment	-	• Differential settlement
1	*U.S. 65 over Pleasant Hill Road		Integral abutment	Pleasant Hill Road	• Void under bridge approach
1	*7781.2R065	1994	Non integral abutment	Four Mile Creek & IAIS Rail Road	• Differential settlement • Settlement of the bridge embankment
1	7778.1L065	1997	Non Integral abutment	6th Avenue	No major problem observed
1	*8561.5L030	1963 Steel bridge 1997 Concrete bridge	Non integral abutment	East Indian Creek	• Soil erosion of the bridge embankment
1	*8556.8L030	1963	Non integral abutment	Union Pacific Rail Road	• Soil erosion of the bridge embankment

Table 3.1 (Continued)

1	*HW 30 over South Skunk River	-	Non integral abutment	South Skunk River	• Soil erosion of the bridge embankment • End drain blocked
1	HW 30 over Duff	-	Non integral abutment	Duff Avenue	• Damage to expansion joint • Differential settlement
1	HW 30 over 136	-	Non integral abutment	136 Road	• Differential settlement
1	South Dakota Bridge	-	Integral abutment	U.S. 30	• Differential settlement
1	Dayton Ave. Bridge	-	Integral abutment	U.P. Rail Road	• Differential settlement
1	*72nd Bridge	-	Integral abutment	I-35	• Differential settlement • Void under approach slab • Transverse cracks at asphalt overlay
1	*Bridge at 160 over I-35	-	Integral abutment	I-35	• Differential settlement • Settlement of the bridge embankment • Lateral movement of the abutment • Expansion joint not sealed
2	1293.7S003	1984	Integral abutments	Hart Groves Creek	• Differential settlement at bridge approach • Cracking expansion joint
2	*1270.2S014	-	-	IANR Rail Road	• Soil erosion of the bridge embankment • Cracking at the approach slab and expansion joint
2	3412.6L018	1999	Integral abutment	I & M Rail Road link	• Differential settlement • Cracking of bridge deck

Table 3.1 (Continued)

2	*3414.4R018	-	Integral abutment	Water stream	• Void under the bridge approach slab • Soil erosion of the bridge embankment • Ponding of water on the bridge embankment
2	3412.3L018	1999	North bound lane approach slabs are supported on geopeirs while the south bound lane approach slabs are supported on fill material. Integral abutment	ICE Rail Road	• Differential settlement • Cracking of bridge deck
2	*3496.7L018	-	Integral abutment	Flood Creek	• Differential settlement • Concrete deterioration at expansion joint
2	1783.6L018	1997	Integral abutment	Union Pacific Rail Road	• Differential settlement • Settlement of the bridge embankment • Lateral movement of the abutment
2	1788.1R035	1972	Integral abutment	Beaver Dam Creek	• Longitudinal cracks of bridge deck

Table 3.1 (Continued)

2	*1791.7R035	1976	Integral abutment	County Road B35	• Erosion along the abutment sides
2	*1793.6R035	1970	Integral abutment	City Street	• Significant damage to concrete slope protection • Lateral movement of the abutment
2	*5596.2S169	1971	Integral abutment	E. Fork Des Moines River	• Settlement of the bridge embankment • Restraining bridge expansion causing cracks
2	*5592.8S169	1971	Integral abutment	East Fork of the Des Moines River	• Cracks at the expansion Joint • Restraining bridge expansion causing cracks • Soil erosion along the abutment sides
2	5588.3S169	1971	Integral abutment	East Fork of the Des Moines River	• Restraining bridge expansion causing cracks
2	*9962.8S003	1976	Integral abutment	Iowa River	• Cracking to bridge approach • Cracks observed at both ends of the bridge • Settlement of the bridge embankment
3	6723.5	1960	Integral abutment	-	• Wavy approach slabs • Differential settlement • Cracking of bridge deck
3	*9701.8	1977-1978	Non integral abutment	S. Lake Port Road	• Water management problem around the bridge

Table 3.1 (Continued)

3	*9701.3	-	Integral abutment	Elk Creek Road	• Damaged expansion joint • Differential settlement • Settlement of the bridge embankment
3	*9702.9	-	Integral abutment	Sunny Brooke Road	• Differential settlement • Damage of concrete slope protection • Settlement of the bridge embankment
3	*9702.0	-	Non integral abutment	Morning Side Road	• Surface drain blocked by soil sediments • Erosion of the bridge embankment
3	*95.35	-	integral abutment	U.S. 75 new bypass	• Void under the approach slab
3	99	2000	Non integral abutment	-	• Strip seal cut short
3	99.5	-	Integral abutment	Rail Road	No major problem observed
3	9798.0	60's	Non integral abutment	Floyd River	• Cracks at the approach caused by pressure relief joints
3	I-29 Border bridge	-	Non integral abutment	Big Sioux River	No major problem observed
3	*West Fork Little Sioux River Bridge	-	-	West Fork Little Sioux River	• Differential settlement • Erosion of the bridge embankment

Table 3.1 (Continued)

4	*786.3R080	-	3 span stub abutment with steel girders	Franklin Road	• Erosion of the embankment and exposed piles • Differential settlement • Damage of expansion joint
4	*786.8R080	-	3 span stub abutment with steel girders	Mc Phearson Ave.	• Settlement of the bridge embankment • Differential settlement • Damage of expansion joints
4	*788.5R080	1968	4 span stub abutment	Hwy 6	• Damage of bridge approach • End drain filled with soil sediments • Cracked embankment cover
4	*7813.1R080	1966-1970	3 span stub abutment with steel girders	Road	• Erosion of the bridge embankment • Differential settlement
4	*7818.6R080	-	Integral abutment with concrete girders	Road	• End drain filled with soil
4	*7821.1L680	-	3 span stub abutment with concrete girders	Co Hwy L34	• Erosion and settlement of the bridge embankment • Damage of expansion joint • Differential settlement

Table 3.1 (Continued)

4	*78071.5R29	-	Integral abutment with concrete girders	I-680	• Damage of expansion joint • Erosion of the bridge embankment exposing H-piles • Water ponding at the bottom of the embankment
4	4375.5R29	-	4 span stub abutment with concrete girders	US Hwy 30	• Damage of the asphalt patch overlay
4	*0184.8L080	-	3 span stub abutment with concrete girders	Middle River	• Cracking of approach slab • Soil erosion of the bridge embankment • Differential settlement
4	*25104.0R080	-	3 span stub abutment with concrete girders	Co Hwy P57	• Significant damage of bridge approach • Differential settlement • End drain filled with soil
4	25110.1R080	-	4 span stub abutment	US Hwy 169	• Differential settlement • Significant damage at the expansion joints
5	*Clay St. over I-35	-	Integral abutment	I-35	• Differential settlement • Void under the approach slab
5	US 218 over Henry St.	-	Integral abutment	Henry St.	• Shearing of paving notch • No subdrain
5	*US 218 over South Skunk River	-	Non integral abutment	South Skunk River	• Erosion of the bridge embankment • Strip seal cut short

Table 3.1 (Continued)

5	*US 218 near exit 42	-	Integral abutment	-	• Poor expansion joint • Void under the approach slab
5	*9266.2R	-	Integral abutment	Creek	• Erosion of the bridge embankment • Subdrain not functioning
5	*9265.6L	-	Integral abutment	Creek	• Differential settlement • Missing join filler material • Erosion of the bridge embankment
6	574.2S001	1992	Non integral abutment	Cedar River	• Differential settlement • Strip seal cut short
6	*5742.4R151	1992	Integral abutment	Big Creek	• Differential settlement • Erosion along the sides of the abutment
6	5289.8R218	1983	Integral abutment	Hwy # 921	• Lateral movement of the abutment • Differential settlement
6	*5200.5R380	1971	Integral abutment	Clear Creek	• Damage of expansion joints • Erosion of the bridge embankment • Ponding of water at the bridge embankment

Table 3.1 (Continued)

6	*5200.8R380	1970	Non integral abutment	US # 6	• Damage to slope protection • Settlement of the bridge embankment • Lateral movement of the abutment
6	5718.4	1972	Integral abutment	Hwy 380	• Differential settlement • Cracks of approach slab • Damage of expansion joint
6	*5200.0R380	-	-	-	• Erosion at the bridge embankment exposing H-piles • Erosion along the abutment sides • Uneven settlement of concrete slope protection

- No data available
*Evidence of water management problem

District 1

Bridge at U.S. 65 over South Skunk River (Mile: 92)

The bridge was constructed in 1999 with non-integral abutments. At the north end of the north bound lane significant settlement was observed. Soil erosion of the embankment under the bridge was also observed (See Figure 3.4). Moreover, concrete spalling was noted at the end of the bridge.

At the south end of the north bound lane (NBL) soil erosion of the bridge embankment was observed. Rocks were later placed to prevent further erosion of the bridge embankment. Concrete deterioration at the end of the girders was noted (See Figure 3.5). The average width of the expansion and the 'CF' joints were 6.5 cm and 9.4 cm respectively.

At the south end of the south bound lane (SBL), lateral movement of the abutment away from the embankment was 2.5 cm. The soil of the bridge embankment was very wet. The differential settlement between the approach slab and the bridge deck was approximately 6.4 cm as shown in Figure 3.6. The width of the expansion joint was 12.7 cm and 13.4 cm at the north and south ends, respectively, and the width of the 'CF' joint was 10.4 cm and 11.2 cm at the north and south ends, respectively.

The bridge approach profiles of both ends of the north bound lane were obtained in spring 2003 (See Figure 3.7). The profiles shows that the south end approach slab experienced more settlement compared to the north end approach slab. Further, the gradients of both the south and north ends are -0.0071 and -0.0051 (i.e. sloping down), which is an indication that either the embankment soil or the foundation soil are settling.

To investigate and characterize the embankment soils, which was 6 m deep, Standard Penetration Test (SPT) was conducted at the north end of the north bound lane (See Figures 3.8 and 3.9). The soil was classified as loose to medium dense sand with a moisture content varying from 12% to 15% up to a depth of 7 m below which the soil was classified as medium to dense sand. The SPT blow counts ranged from 5 at the surface to 20 at about 10.6 m with the water table at a depth of 7.6 m. Therefore, it is more likely that the embankment material has undergone compression, which was observed in the north bound lane profiles.

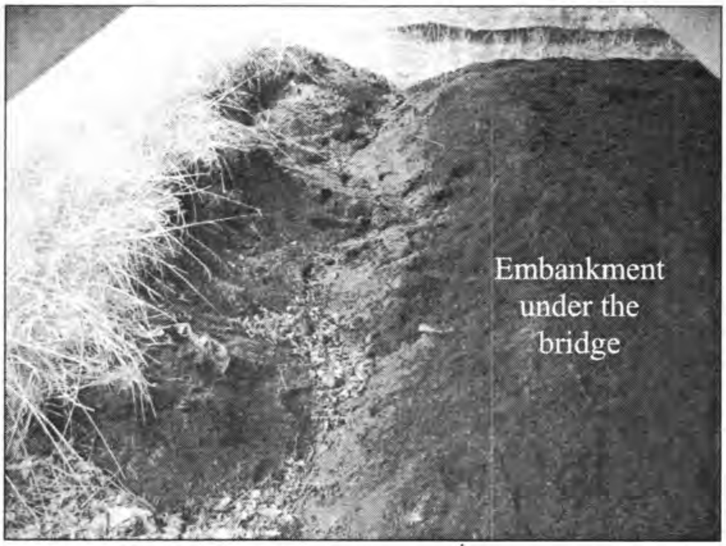


Figure 3.4 - Soil Erosion of the Bridge Embankment (Bridge at U.S. 65 over South Skunk River - South End of NBL)

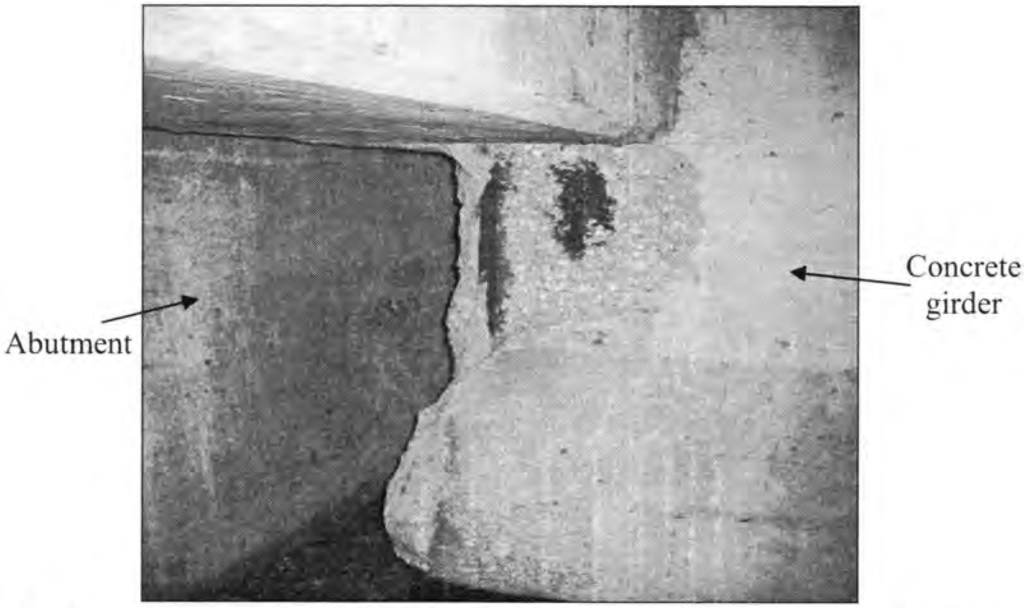


Figure 3.5 - Deterioration of Concrete with Visible Steel Reinforcement (Bridge at U.S. 65 over South Skunk River - South End NBL)

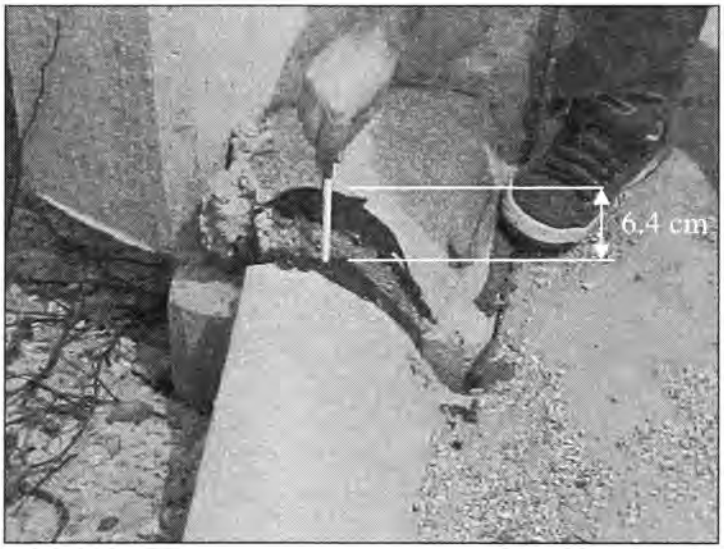


Figure 3.6 - Differential Settlement between the Bridge Beck and the Approach Slab (Bridge at U.S. 65 over South Skunk River - South End SBL)

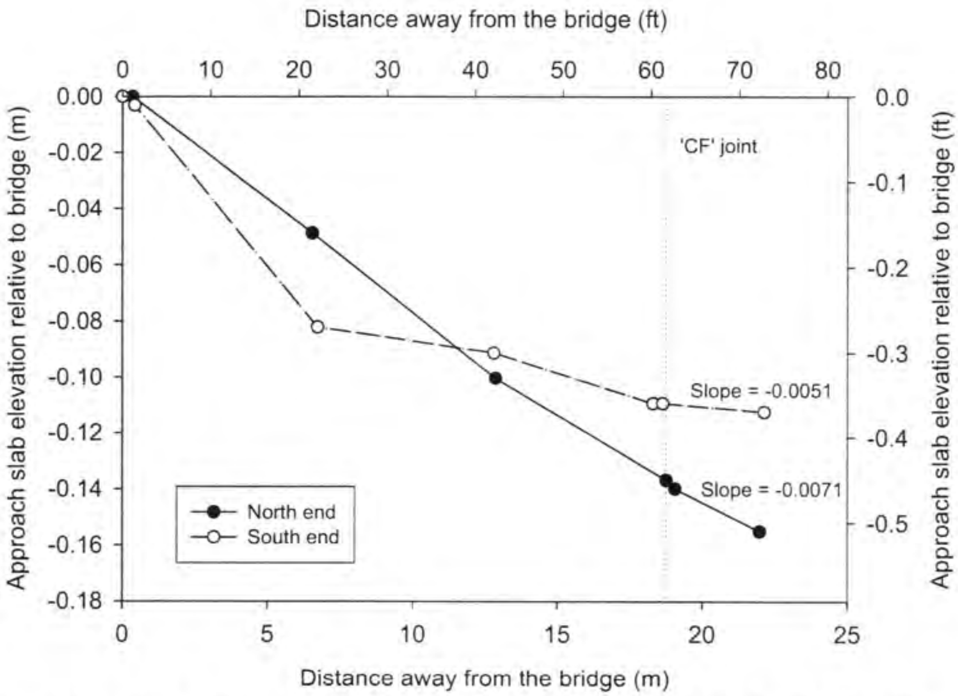


Figure 3.7 - Elevation of Bridge Approaches Relative to Bridge Deck (Bridge at U.S. 65 over South Skunk River - NBL)

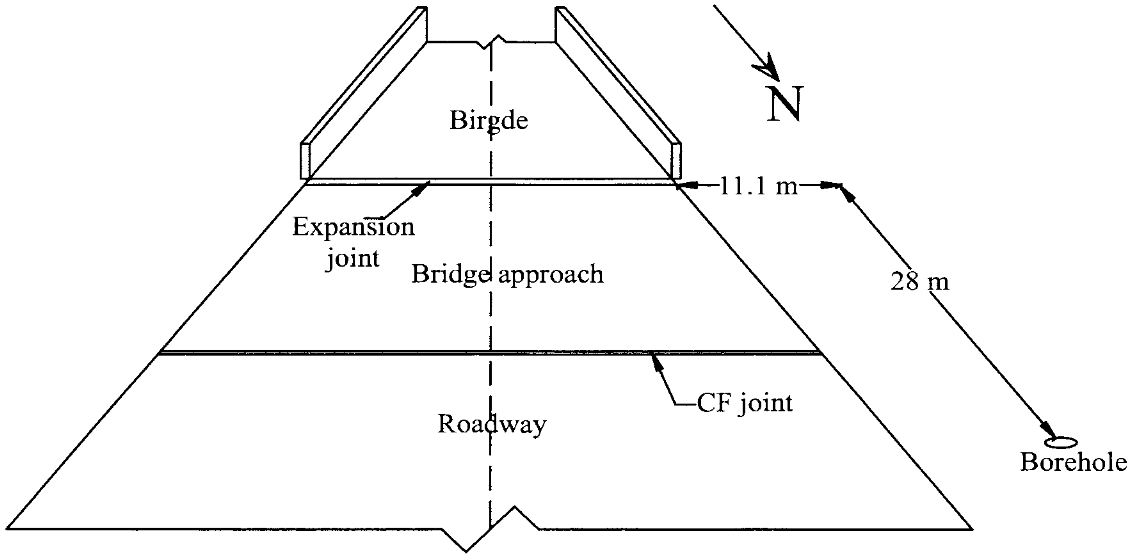


Figure 3.8 - Location of Test Borehole (Bridge at U.S. 65 over South Skunk River)

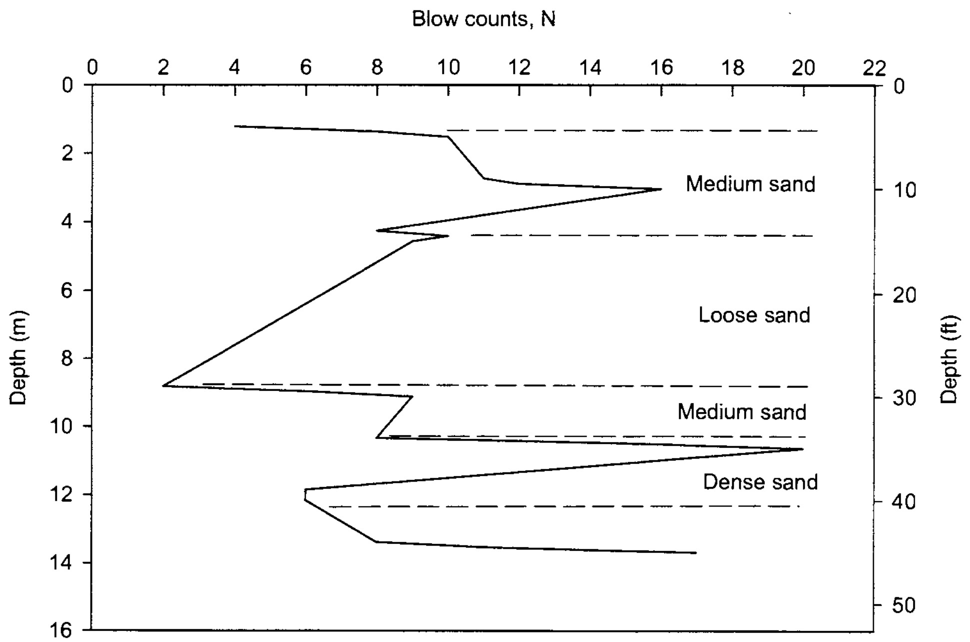


Figure 3.9 - Results of SPT and Classification of the Embankment Soil (Bridge at U.S. 65 over South Skunk River)

<u>Bridge at U.S. 65 over Rail Road (Mile 89)</u>

This bridge has non-integral abutments. Water was observed flowing through all expansion joints downward toward the bridge embankment. Soil erosion noticed at all four ends of the bridge. The severe soil erosion shown in Figure 3.10 was observed at the north end of the north bound lane. At the south end of the north bound lane, the approach slab at the settled approximately 15.2 cm from its original level relative to the wingwall as shown in Figure 3.11. At both the north and south ends of the south bound lane severe erosion and settlement of the embankment was noted. The embankments were constructed using sandy soils which is highly erodible (See Figures 3.12 to 3.14).

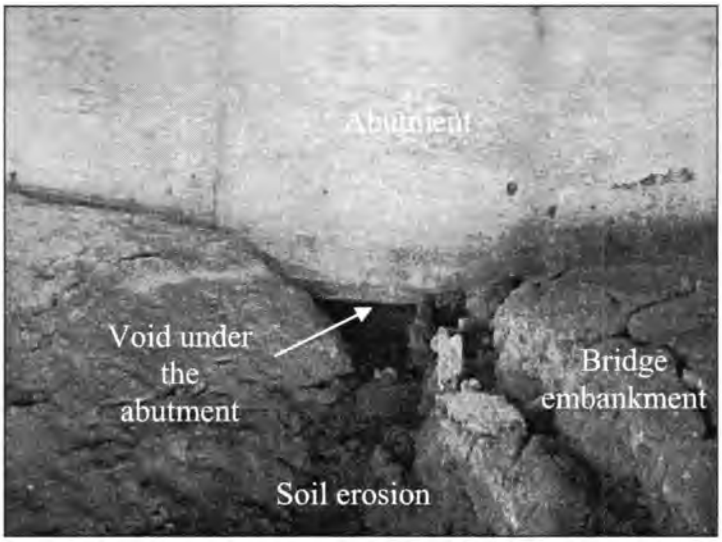


Figure 3.10 - Erosion of Soil Under the Bridge Abutment (Bridge at U.S. 65 over Rail Road - North End of NBL)

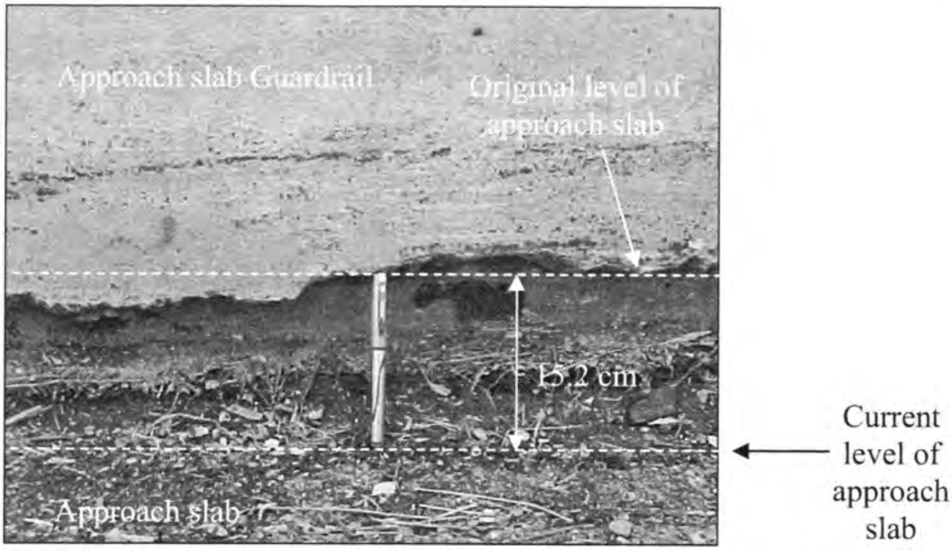


Figure 3.11 - Settlement of Bridge Approach Slab Relative to the Wingwall (Bridge at U.S. 65 over Rail Road - South End of NBL)

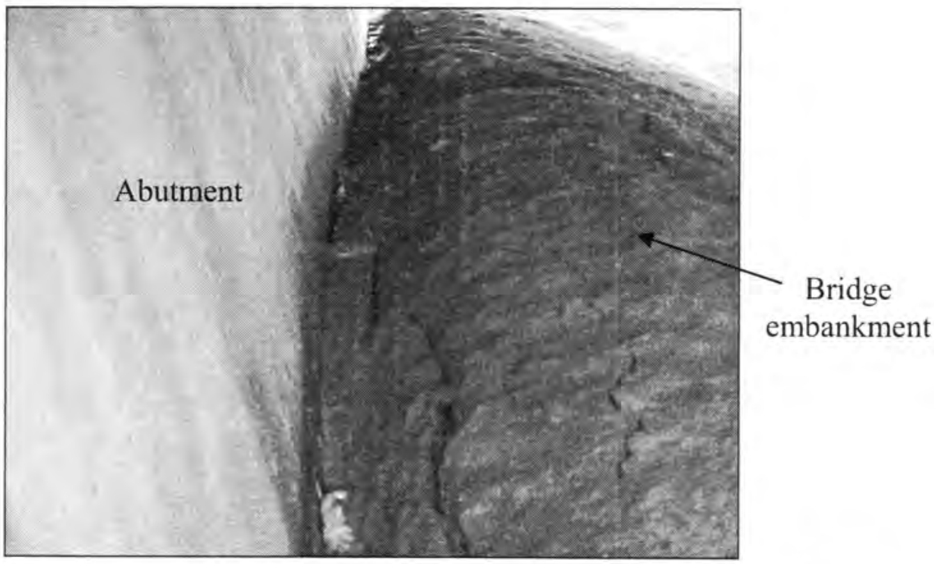


Figure 3.12 - Erosion between the Abutment and the Bridge Embankment (Bridge at U.S. 65 over Rail Road - North End of SBL)

Figure 3.13 - Soil Erosion Under the Bridge (Bridge at U.S. 65 over Rail Road - North End of SBL)

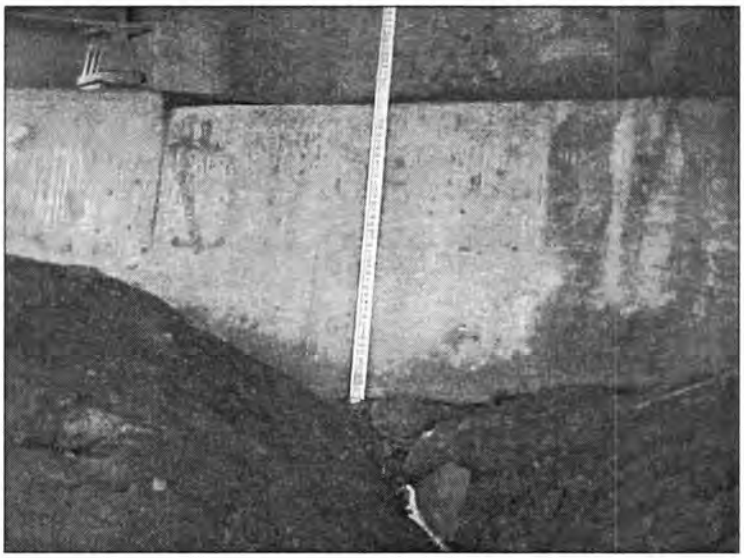


Figure 3.14 - Soil Erosion of the Bridge Embankment (Bridge at U.S. 65 over Rail Road - South End of SBL)

<u>Bridge at U.S. 65 crossing Pleasant Hill Road</u>

The bridge has integral abutments and concrete girders. Aggregate was used under the bridge as a slope protection cover to minimize erosion (See Figure 3.15). However, soil erosion was observed between the embankment and the backwall. Soil erosion was observed at both north and south bound lanes. Furthermore, a 7.6 cm void has developed under the approach slab (See Figure 3.16).

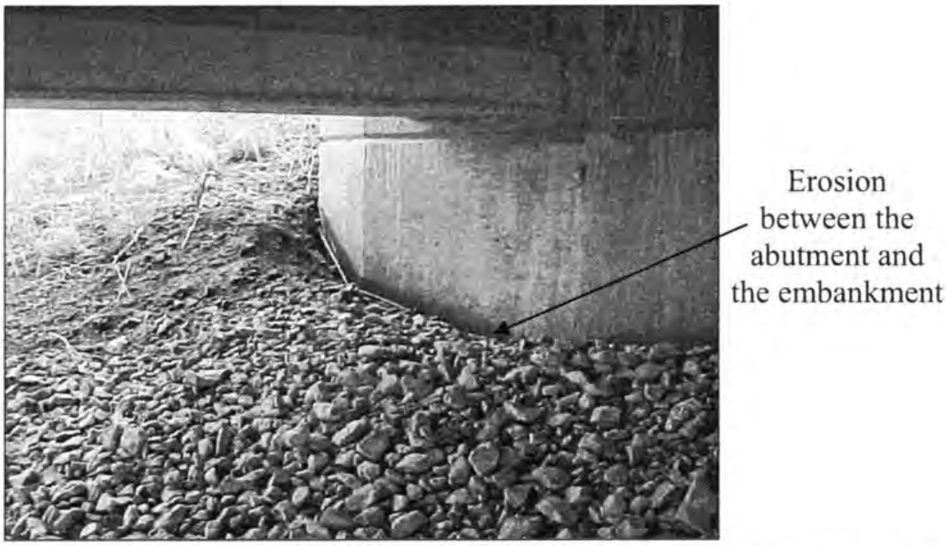


Figure 3.15 - Aggregate Used as Slope Protection at the Bridge Embankment (Bridge at U.S. 65 Crossing Pleasant Hill Road)

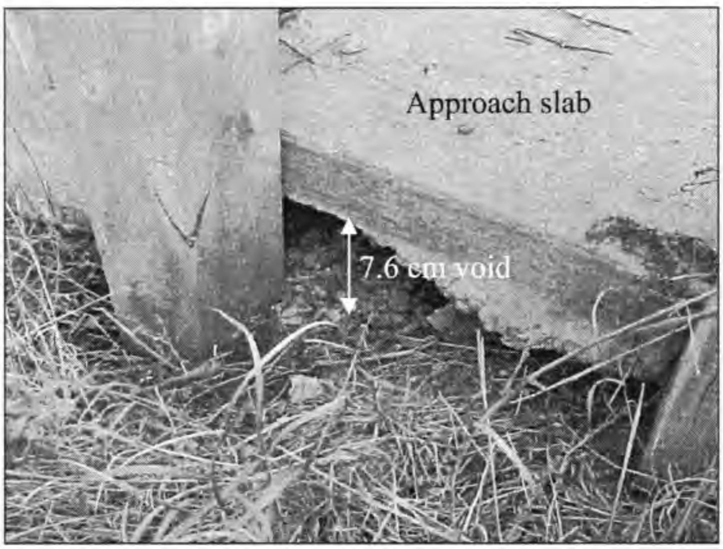


Figure 3.16 - Void Developed Under the Approach Slab (Bridge at U.S. 65 Crossing Pleasant Hill Road - South End of NBL)

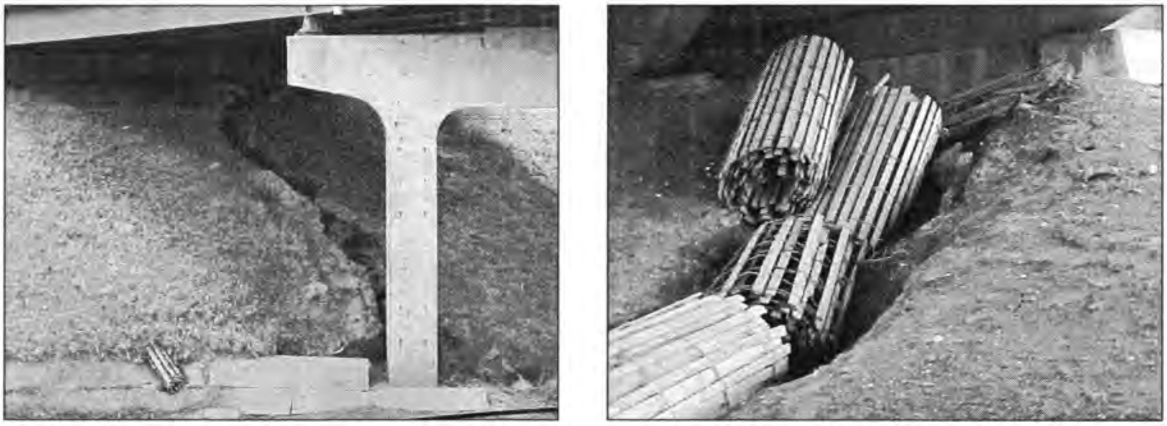
Bridge No. 8561.5L030 (US 30 over East Indian Creek)

The bridge has non-integral abutments. The east bound of the bridge is supported on concrete girders and is fairly new, while the west bound is supported on steel girders and was constructed prior to the east bound. Soil erosion was observed around the bridge as shown in Figure 3.17. No other major problems were noticed.

Figure 3.17 - Soil Erosion of the Bridge Embankment (Bridge No. 8556.8L030)

Bridge No. 8556.8L030 (U.S. 30 over U.P. Railroad)

This bridge has non-integral abutments and steel girders. Significant erosion was observed under the bridge at both the east and the west bounds as shown in Figure 3.18. However, the approach slab appeared to be in a good condition.



(a) East end **(b) West end**

Figure 3.18 - Soil Erosion of the Bridge Embankment (Bridge No. 8556.8L030)

<u>Bridge Carrying 72nd St. over I-35 (South of Hwy 5)</u>

The bridge is a four span bridge with integral abutments and concrete girders. An asphalt overlay was placed at both ends of the bridge to compensate for elevation difference caused by differential settlement (See Figure 3.19). Transverse cracks were observed at the asphalt overlay (See Figure 3.20). Differential settlement at the bridge approach was 7.6 cm measured at the approach slab shoulder. Minor erosion of the bridge embankment and along the abutment sides was noted. Furthermore, the expansion joint was poorly sealed with a width of 10.2 cm at the east end (See Figure 3.21). As shown in Figure 3.22 aggregate was used as a slope protection at the bridge embankment.

Figure 3.19 - Recently Placed Asphalt Overlay at the Bridge Approach (Bridge Carrying 72nd St. over I-35 - East End)

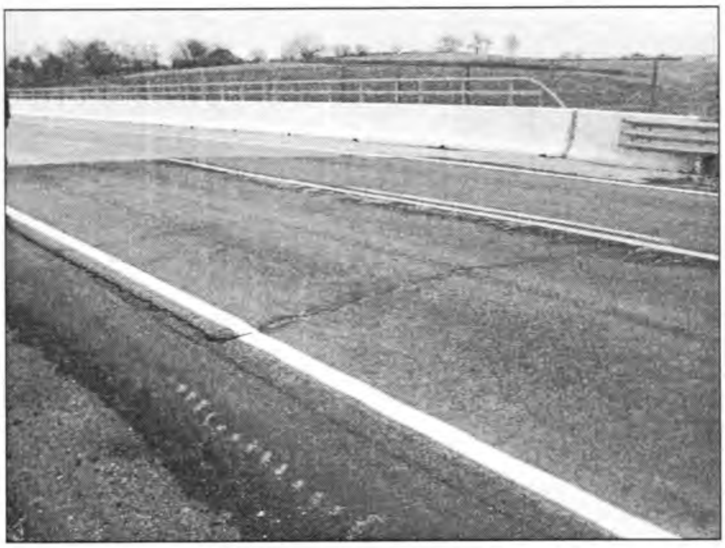


Figure 3.20 - Cracking of the Asphalt Overlay (Bridge Carrying 72nd St. over I-35 - East End)

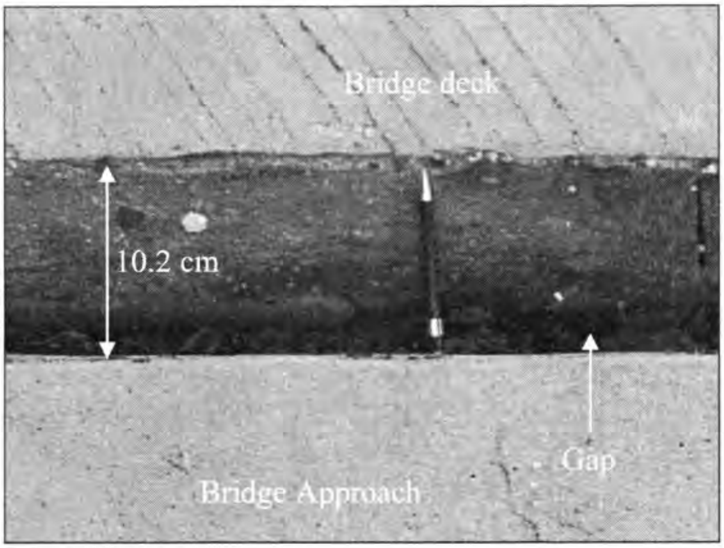


Figure 3.21 - Poorly Sealed Expansion Joint (Bridge Carrying 72nd St. over I-35 - East End)

Figure 3.22 - Aggregate Used as a Slope Protection at the Bridge Embankment (Bridge Carrying 72nd St. over I-35)

The bridge was revisited again in summer 2004. At the east end, settlement of the approach slab relative to the wingwall was 16.5 cm and a void depth of 14 cm under the approach slab was observed. The width of the expansion joint was 14 cm (See Figure 3.23).

More transverse cracks were noted on the asphalt overlay indicating more settlement of the approach slab underneath (See Figure 3.24).

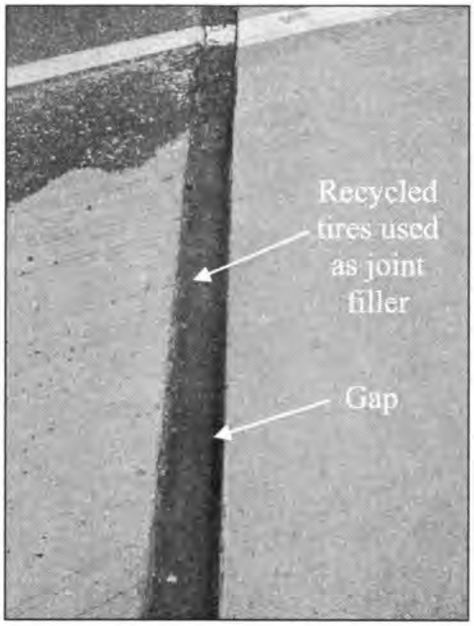


Figure 3.23 - Poorly Sealed Expansion Joint (Bridge Carrying 72nd St. over I-35 - East End)

Figure 3.24 - Transverse Cracks of Recently Resurfaced Approach Slab (Bridge Carrying 72nd St. over I-35 - East End)

At the west end, a void has developed under the approach slab. The void was 14 cm at the expansion joint and 11.4 cm at the end of the wingwall (See Figure 3.25). Furthermore, soil erosion along the abutment side was observed (See Figure 3.26). The soil used near the abutment sides appeared to be silty sand which is highly erodible. The gravel slope protection at the bridge embankment was in good condition.

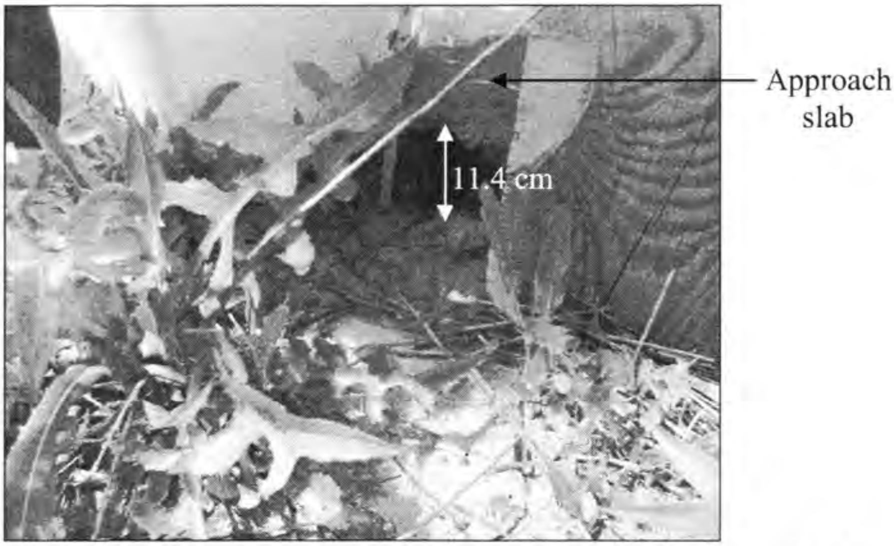


Figure 3.25 - Void Developed Under the Approach Slab (Bridge Carrying 72nd St. over I-35 - West End)

Figure 3.26 - Soil Erosion Along the Abutment Side (Bridge Carrying 72nd St. over I-35 - West End)

<u>Bridge Carrying 160 over I-35</u>

The bridge has integral abutments and concrete girders. Differential settlement between the bridge deck and the approach slab was 3.8 cm at the east end (See Figure 3.27). The bridge embankment has settled 9 cm and a gap of 7.6 cm wide between the abutment and the bridge embankment was measured (See Figure 3.28). Flexible foam was used as a filler material at the expansion joint which was 12.7 cm wide.

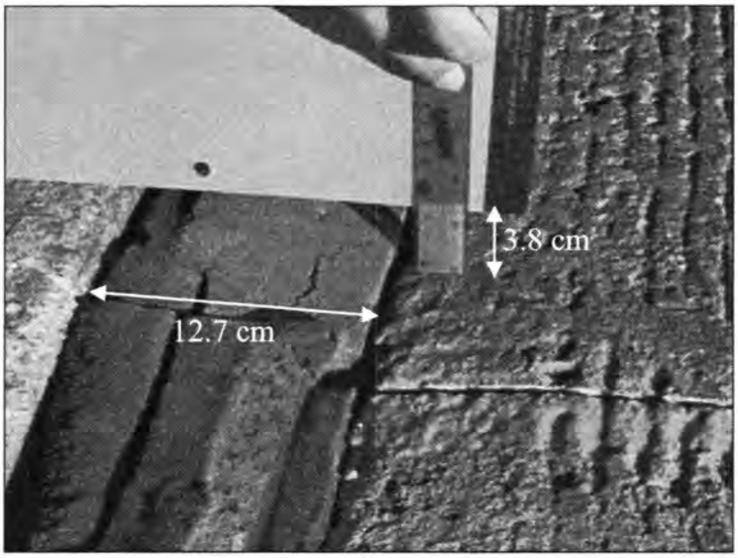


Figure 3.27 - Differential Settlement Between the Bridge Approach and the Bridge Deck (Bridge Carrying 160 over I-35 - East End of WBL)

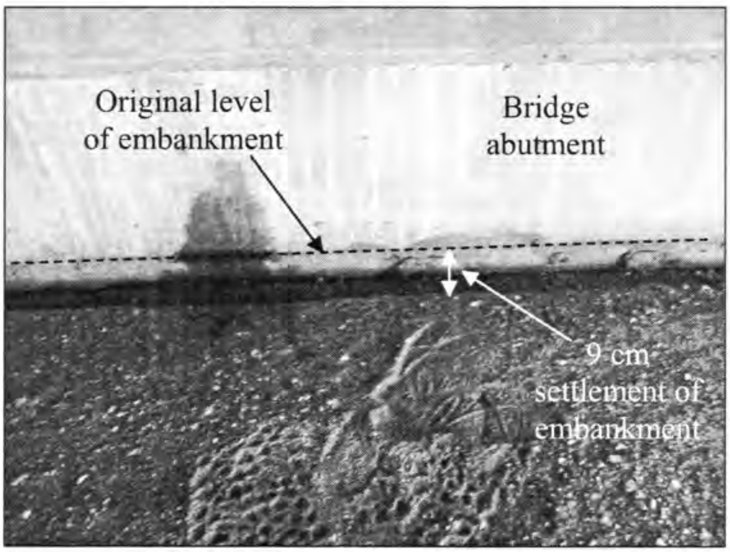


Figure 3.28 - Settlement of the Embankment under the Bridge and a Gap between the Abutment and the Bridge Embankment (Bridge Carrying 160 over I-35 - East End WBL)

At this bridge, part of the approach slab at the east end of the west bound lane was removed to investigate the cause of the differential settlement. The section removed was 0.9 m long and 4.6 m wide extending from the centerline of the approach slab to the center of the shoulder (See Figures 3.29 to 3.31). After removing the approach slab segment, a 20.3 cm void was observed (See Figure 3.32), and the base material appeared to be poorly placed and without compaction. The base material was a mixture of fine and coarse sand. A layer of 3.8 cm thick of fine sand particles was observed over the paving notch, which may be a result of crushed base material left on the paving notch during construction (See Figure 3.33). The thickness of the approach slab was 17.8 cm over the paving notch and 21.6 cm over the approach slab embankment (See Figure 3.34) and the width of the paving notch was 25.4 cm. As shown in Figure 3.35, flexible foam, which poorly sealed the expansion joint, was used as a joint filler material.

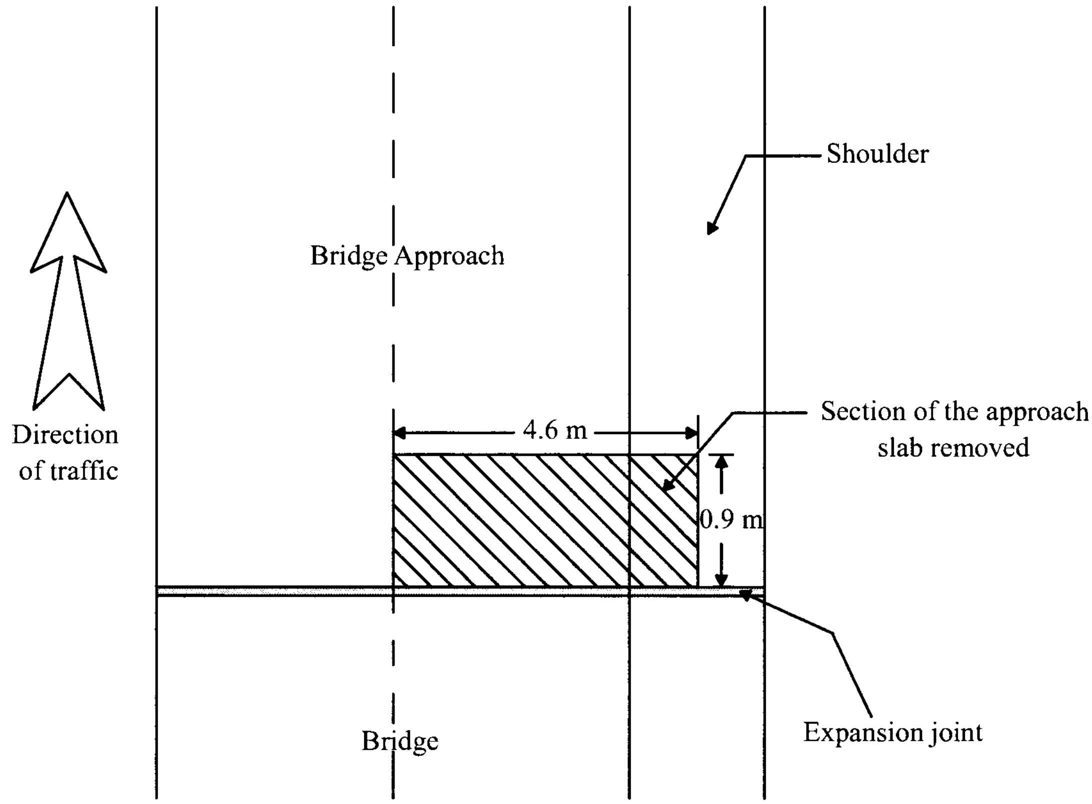


Figure 3.29 - Plan View of the Bridge with the Location of Removed Approach Slab Section for Inspection (Bridge Carrying 160 over I-35)

Figure 3.30 - Section of the Approach Slab Cut and Ready to be Removed (Bridge Carrying 160 over I-35)

Figure 3.31 - Removing a Segment of the Approach Slab (Bridge Carrying 160 over I-35)

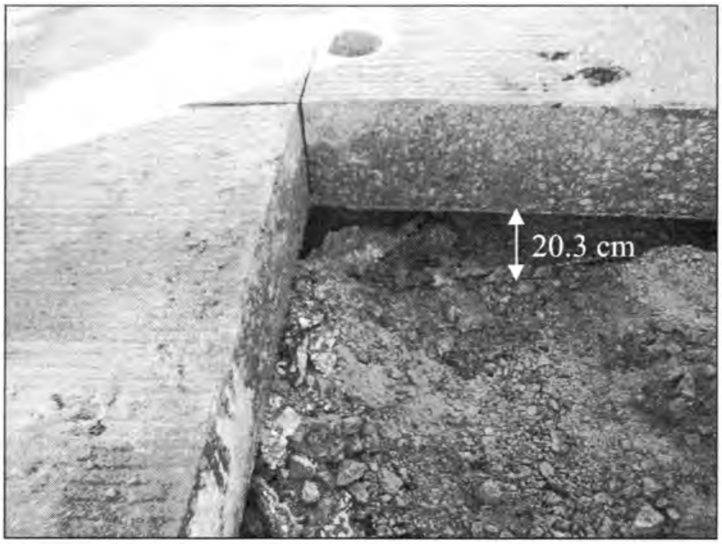


Figure 3.32 - Void Observed under the Approach slab (Bridge Carrying 160 over I-35)

Figure 3.33 - Layer of Crushed Aggregate Covering the Paving Notch (Bridge Carrying 160 over I-35)

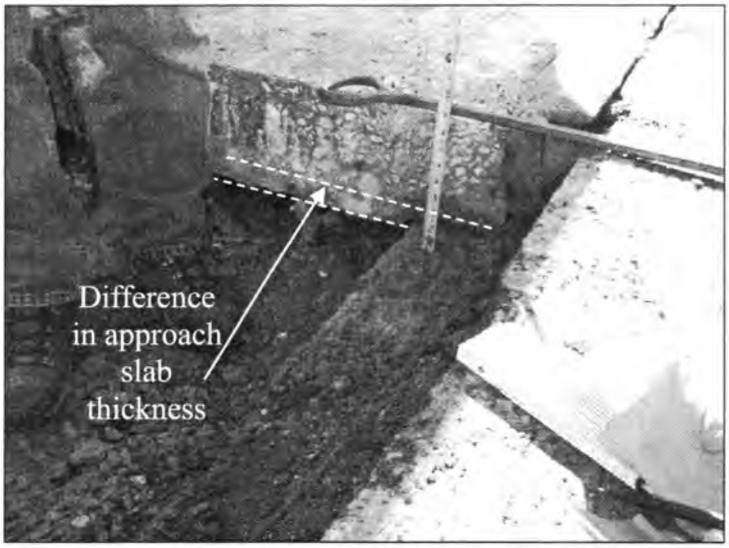


Figure 3. 34 - Non Uniform Approach Slab thickness with 3.8 cm Increase Over the Backfill Compared with the Thickness over the Paving Notch (Bridge Carrying 160 over I-35)

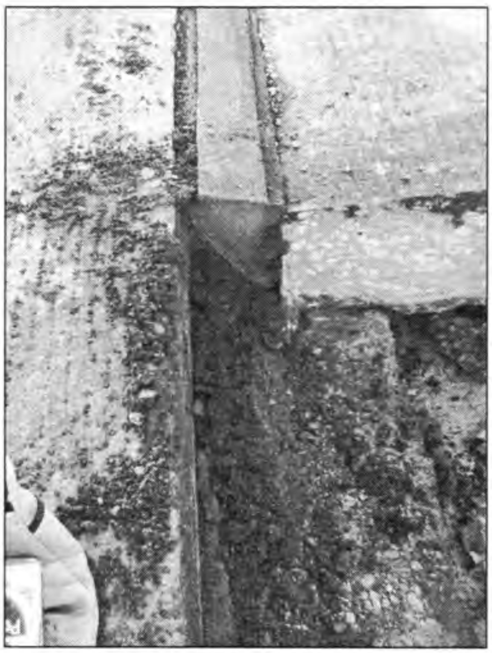


Figure 3. 35 - Flexible Foam Used to Seal the Expansion Joint (Bridge Carrying 160 over I-35)

District 2

Bridge No. 1270.2S014 (Hwy 14 over IANR Railroad)

This bridge is a three span concrete bridge with a deep slab and no girders (See Figure 3.36). Figure 3.37 shows erosion of the bridge embankment. Cracks were observed around the expansion joint which was covered by an asphalt overlay (See Figure 3.38).

Figure 3.36 - Concrete Bridge with Deep Slab and No Girders (Bridge No. 1270.2S014)

Figure 3.37 - Erosion of the Embankment Soil under the Bridge (Bridge No. 1270.2S014)

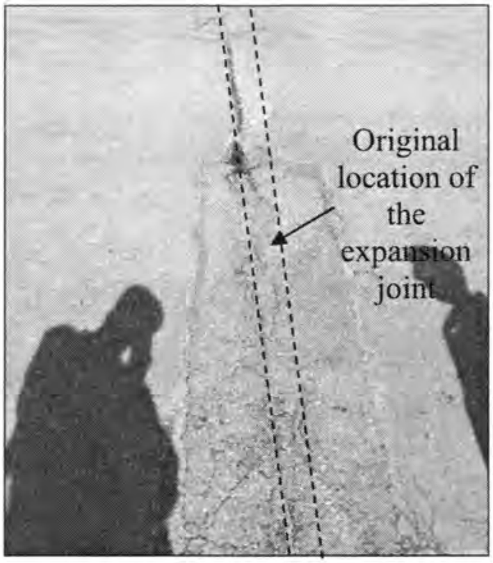


Figure 3.38 - Cracked Asphalt Patch at the Expansion Joint (Bridge No. 1270.2S014)

Bridge No. 3414.4R018 (U.S. 18 over Creek)

This bridge has integral abutments and prestressed concrete girders. A 5.1 cm void had developed under the bridge approach. In addition, erosion of the embankments under the bridge was observed. Figure 3.39 shows water ponding between the embankment and the abutment which is mainly caused by improper drainage around the bridge.

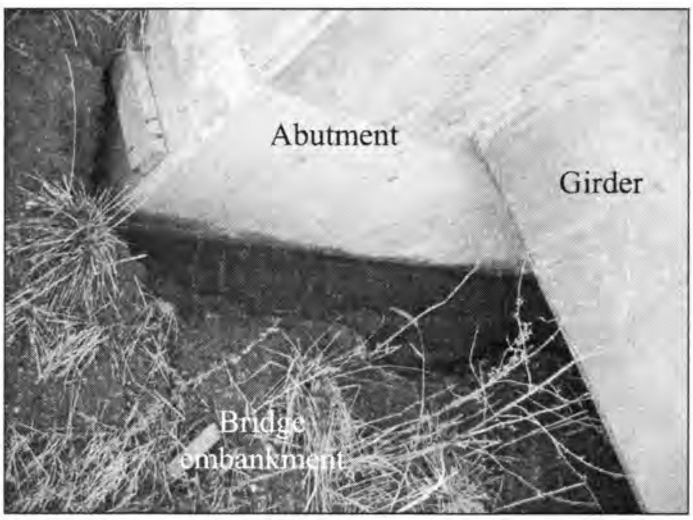


Figure 3.39 - Water Ponding on the Embankment under the Bridge (Bridge No. 1270.2S014)

Bridge No. 3412.3L018 (Hwy 18 over Railroad)

This bridge has a concrete bridge with integral abutments, which was constructed in 1999

One previous attempt made by Iowa DOT in collaboration with Iowa State University to alleviate the settlement of the approach slab was to use Geopier foundation elements to support the approach slab. A total of 30 Geopier elements spaced at 1.8 m center to center both directions were used at each approach slab of the north bound lane. Six rows of Geopier elements with the first row at approximately 1.2 m from the edge of the driven H-piles were constructed as shown in Figure 3.40 (White *et al.* 2003). The settlements of the approach slabs at all four ends of the bridge were monitored for 42 months. Figures 3.41 through 3.44 show the monitored settlement of all approach slabs.

Figures 3.41 and 3.42 show the settlement of the bridge approaches supported on Geopier elements at both the north and south ends of the north bound lane, respectively. Settlement data of both approaches indicate that the approach slab settlement has been increasing with time.

Figures 3.43 and 3.44 show the settlement of the bridge approaches supported on fill material of the north and south ends of the south bound lane, respectively. It is observed that approach slab settlement increases with time at both approach slabs. Figures 3.41 through 3.44 show no significant settlement difference between approach slabs supported on Geopiers elements or embankment fill. This indicates that the Geopiers constructed under the north bound lane was not effective in alleviating the bridge approach settlement. A possible improvement for this method is to extend the geopier elements deeper in the foundation soil.

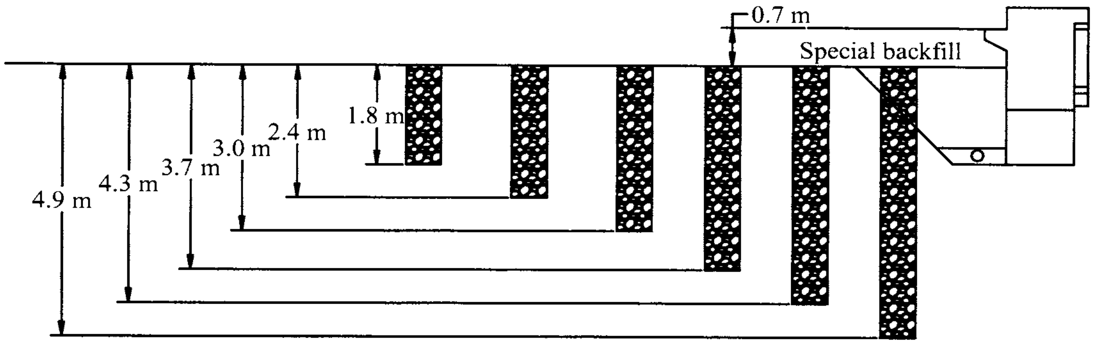


Figure 3.40 - Profile of Geopier Elements Supporting Bridge Approach (White *et al.* 2003)

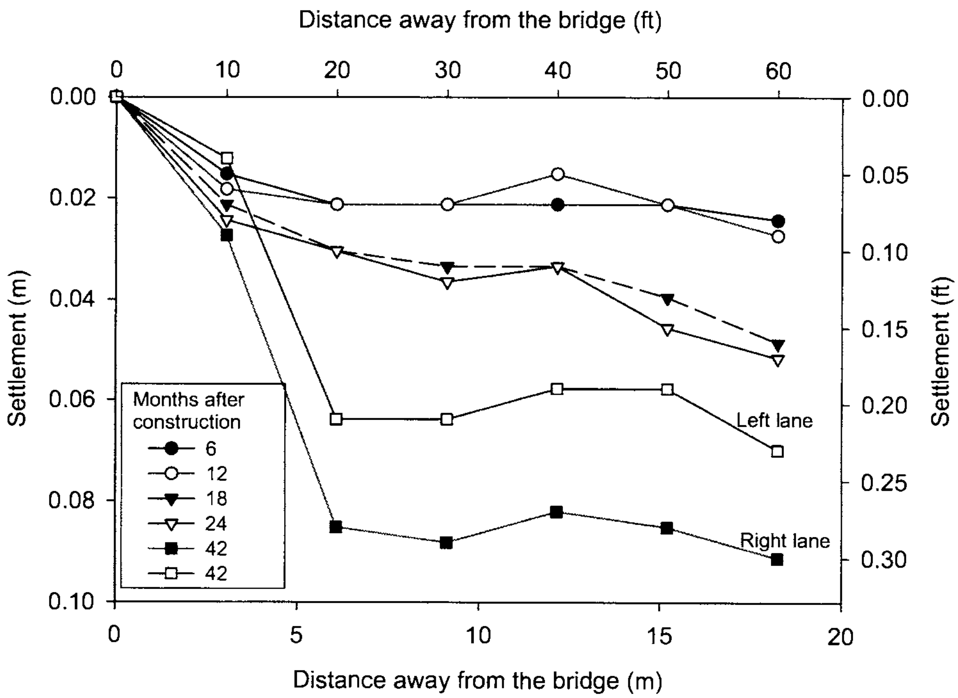


Figure 3.41 - Settlement of the Aapproach Slab Supported on Geopier Elements (Bridge No. 3412.3L018 - North End of NBL)

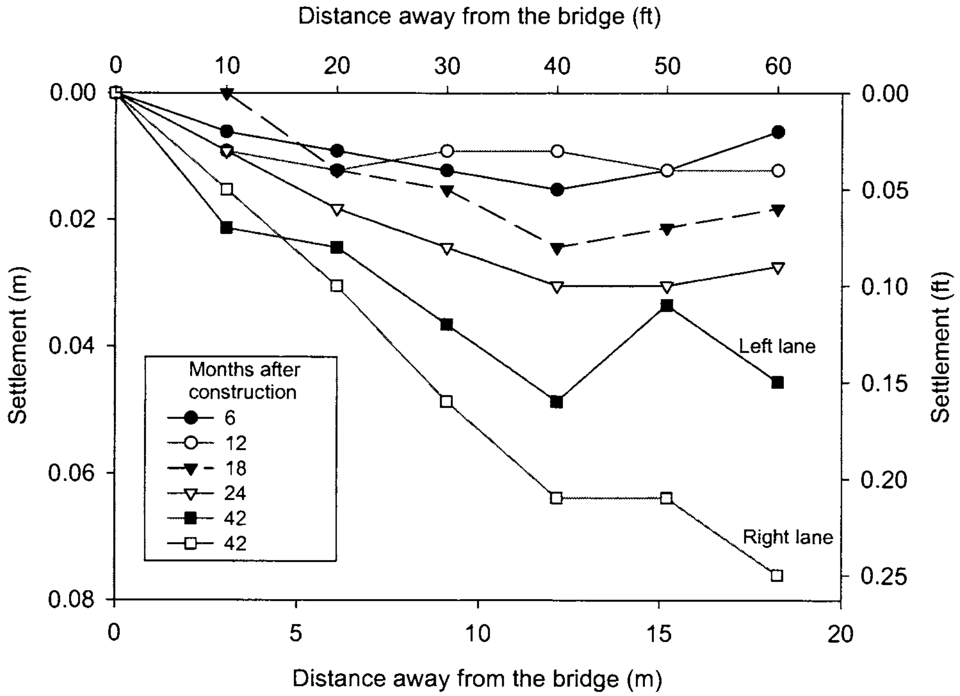


Figure 3.42 - Settlement of the Approach Slab Supported on Geopier Elements (Bridge No. 3412.3L018 - South End of NBL)

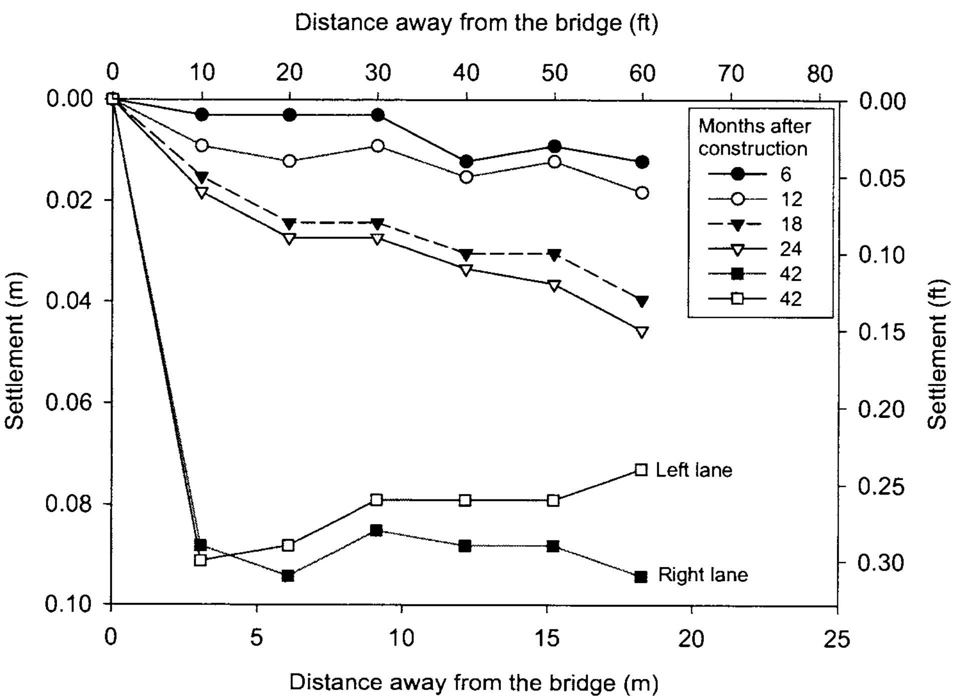


Figure 3.43 - Settlement of Bridge Approach Supported on the Embankment Soil (Bridge No. 3412.3L018 - North End of SBL)

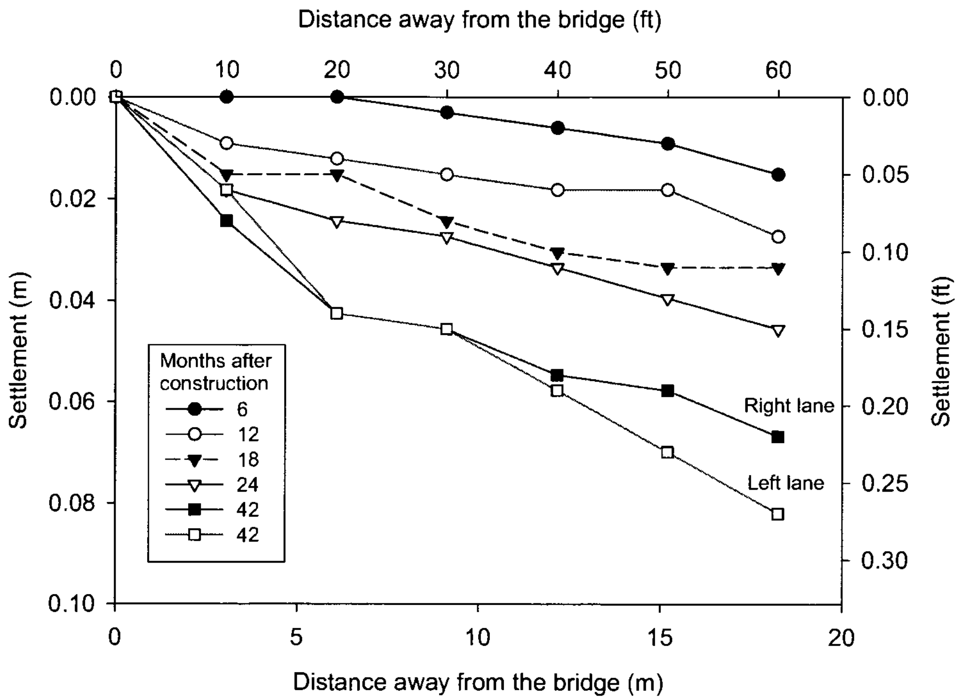


Figure 3.44 - Settlement of Bridge Approach Supported on Embankment Soil (Bridge No. 3412.3L018 - South End of SBL)

Bridge No. 1793.6R035 (I-35 over City Street)

The bridge approach was resurfaced using an asphalt overlay. The bridge embankment had a concrete slope protection cover. Figure 3.45 and 3.46 show the damage of the concrete slope protection at the south end of the north bound lane. The depth of the void under the concrete slope protection was approximately 0.5 m. Figure 3.47 shows both the settlement of the bridge embankment and the horizontal movement of the abutment away from the embankment. The width of the gap between the abutment and the embankment was 16.5 cm.

(a) Damaged Slope Protection

(b) Closer Image of the Damaged Concrete

Figure 3.45 - Failure of Slope Protection Due to Loss of Support (Bridge No. 1793.6R035 - South End NBL)

Figure 3. 46 - Fractured Concrete Slope Protection (Bridge No. 1793.6R035)

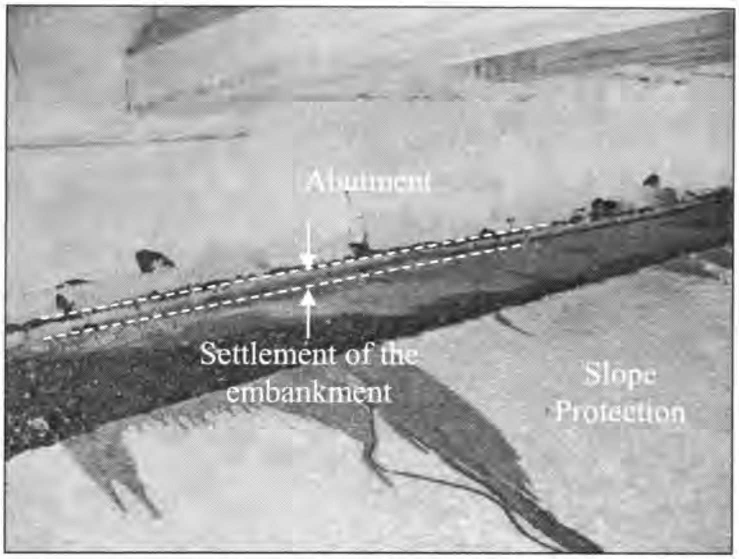


Figure 3. 47 - Settlement of the Bridge Embankment and a Gap between the Embankment and the Abutment (Bridge No. 1793.6R035)

Bridge No. 5596.2S169 (Hwy 169 over E. Fork Des Moines River)

This bridge, which was constructed in 1971, has integral abutments and concrete girders. According to the maintenance report, grout was pumped at the bridge ends to fill the void under the approach slab. However, when hardened, the grout prevented the lateral movement of the bridge abutment. This caused the bridge deck to break as shown in Figure 3.48. Bridge approach profiles relative to the bridge deck were obtained in summer 2003 (See Figure 3.49). A settlement of approximately 5.1 cm was measured between the approach slab and the roadway at the north end approach slab. The south end approach slab settled a distance of approximately 10.2 cm between 5 and 15 m away from the bridge. The gradient of the north and south ends are 0.003 and -0.008 respectively indicating that the approach slabs are sloping towards and away from the bridge. Figure 3.50 shows the inlet of the bridge end drain which appeared to be effective since no soil erosion around the bridge was observed.

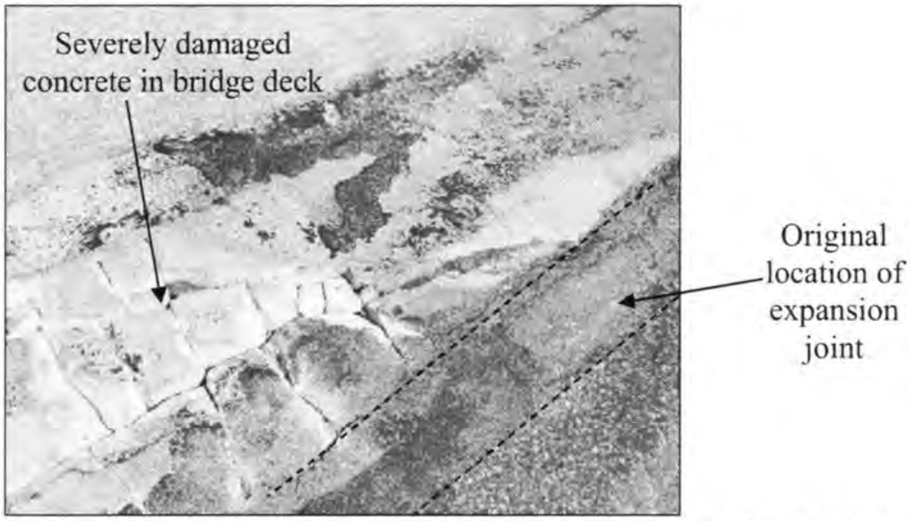


Figure 3. 48 - Bridge Deck Damage was suspected to be a Result of Grouting behind the Bridge Abutment (Bridge No. 5596.2S169)

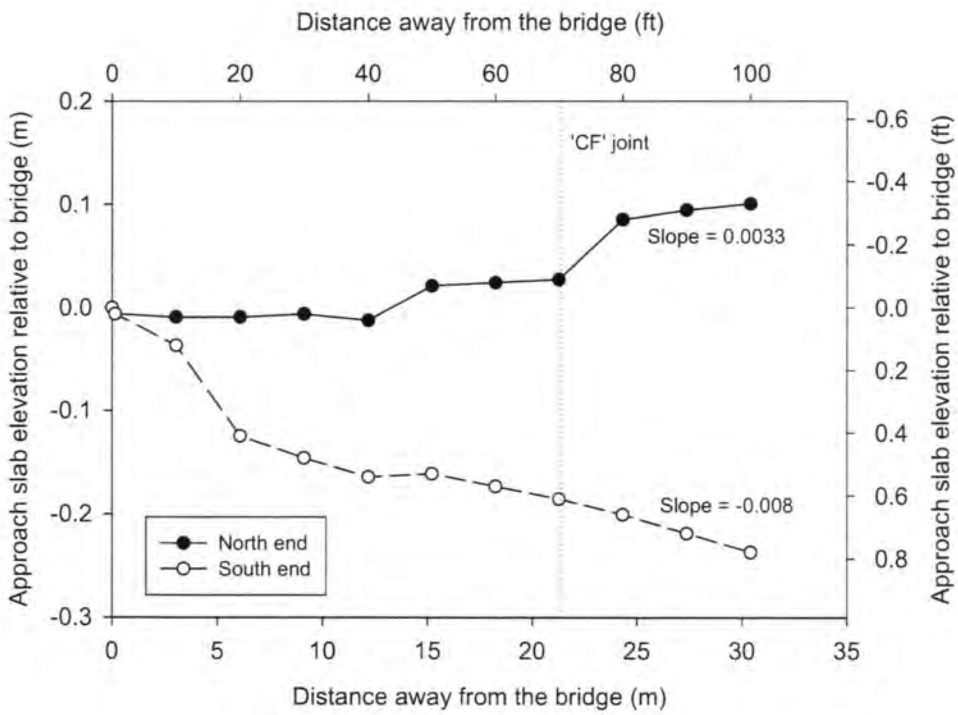


Figure 3. 49 - Elevation of Approach Slabs Relative to the Bridge (Bridge No. 5596.2S169)

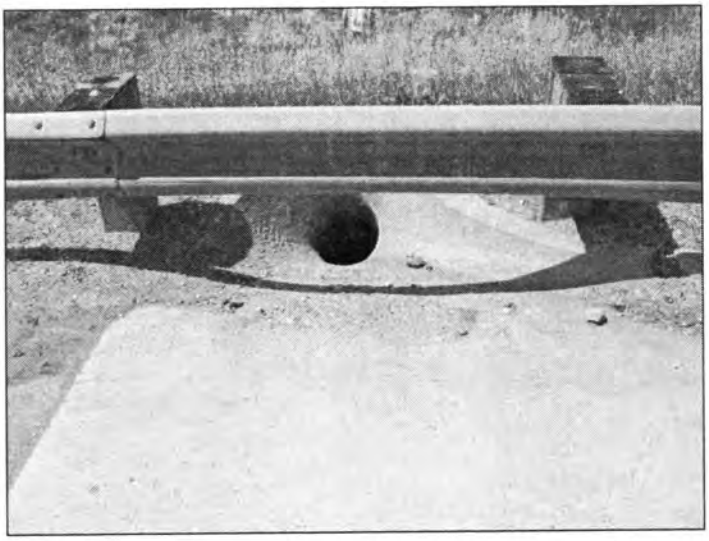


Figure 3. 50 - Bridge End Drain Inlet (Bridge No. 5596.2S169)

District 3

Bridge No. 9702.9 (U. S. 20 East over Sunny Brooke Road)

This bridge has integral abutments and concrete girders. The bridge embankment had slope protection. The expansion join at the bridge was 5.1 cm wide. Figure 3.51 shows the 10.2 cm differential settlement measured between the bridge deck and the bridge approach at the approach slab shoulder. The figure also shows alligator cracking at the bridge shoulder. Grout was pumped behind the abutment at the west end of the east bound lane and under the embankment overlay. The slope protection settled about 16.5 cm (See Figure 3.52 and 3.53), which was caused by loss of support due to erosion. Lateral movement of the abutment away from the embankment was also observed.

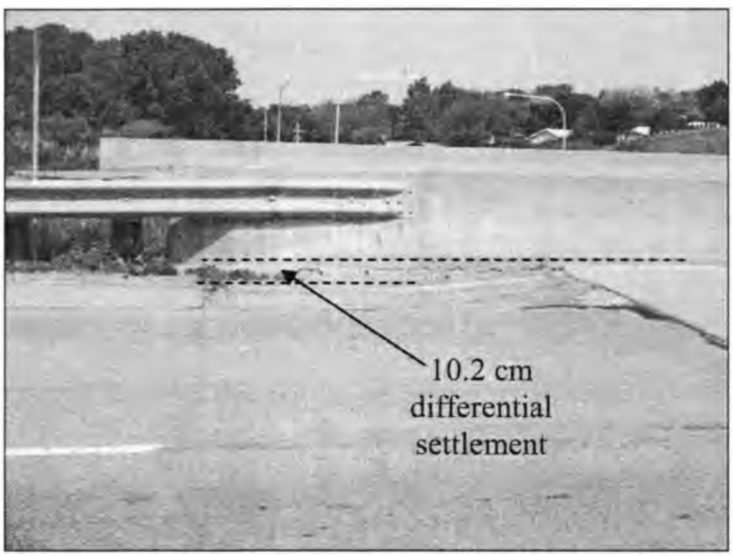


Figure 3.51 - Differential Settlement between the Bridge Deck and the Approach Slab (Bridge No. 9702.9)

Figure 3. 52. Uneven Settlement of slope protection (Bridge No. 9702.9)

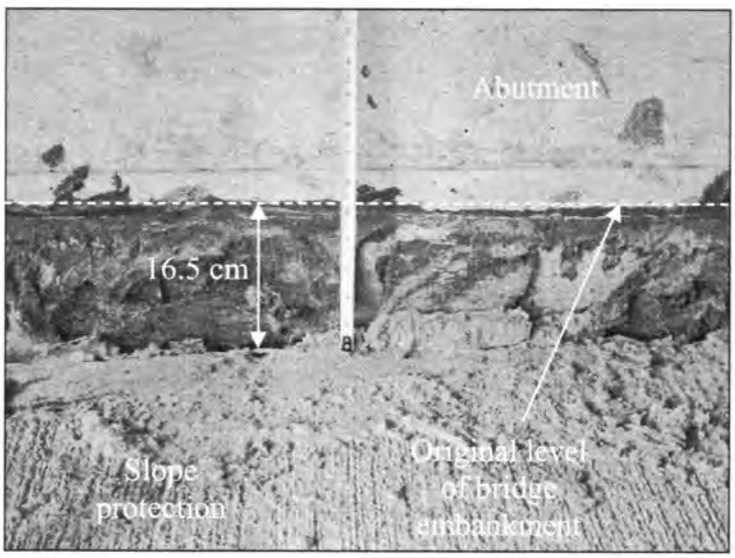


Figure 3. 53. Settlement of the Bridge Embankment (Bridge No. 9702.9)

<u>Bridge No. 982.9 (U.S. 20 over Morning Side Road)</u>

This bridge has non-integral abutments and concrete girders. The joint was grouted to prevent water from flowing down the bridge embankment. This caused the water to run along the bridge shoulder towards the other end of the bridge. Although s bridge end drain was used on the west side of the bridge, it was blocked with soil sediments (See Figure 3.54). As a result running water eroded the approach slab shoulder as shown in Figure 3.55. With no slope protection under the bridge and silty soil used as the embankment soil, sever erosion was also observed (See Figure 3.56).

(a) Bridge End Drain Blocked by Debris

(b) Bridge End Drain After Debris Removal

Figure 3.54 - Drainage Intake Before and After Debris Removal (Bridge No. 982.9)

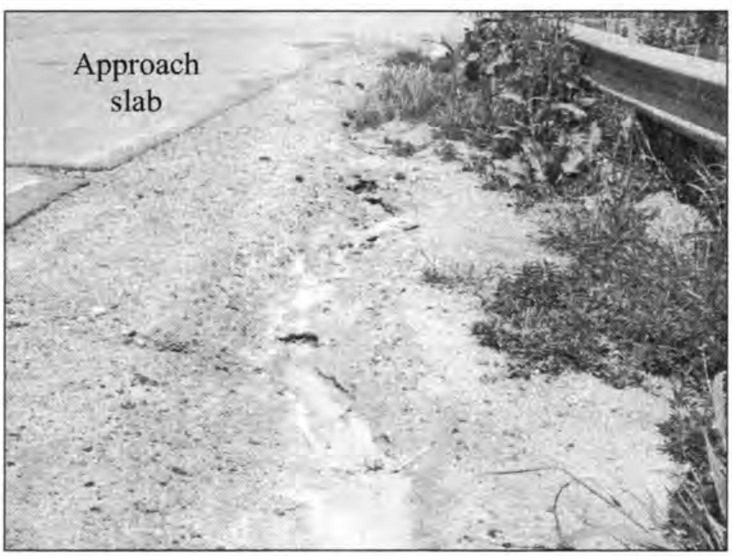


Figure 3.55 - Erosion Caused by Runoff Water at the Approach Slab Shoulder (Bridge No. 982.9)

Figure 3.56 - Severe Erosion under the Bridge (Bridge No. 982.9)

Bridge No. 99 (U.S. 20 over Highway 75)

The bridge was constructed in 2000 with non-integral abutments. The foundation soil at this bridge was allowed to consolidate under the embankment weight for one year prior to construction. No differential settlement was observed at the approach slabs. Erosion of the soil under the bridge was observed because the strip seal of the expansion joint was cut short at both sides of the north bound lane causing water to run down the sides of the bridge (See Figures 3.57 and 3.58). At the opposite end of the bridge the same strip seal problem was noticed and erosion of the embankment soil was observed.

Figure 3.57 - Soil Erosion of the Bridge Embankment (Bridge No. 99)

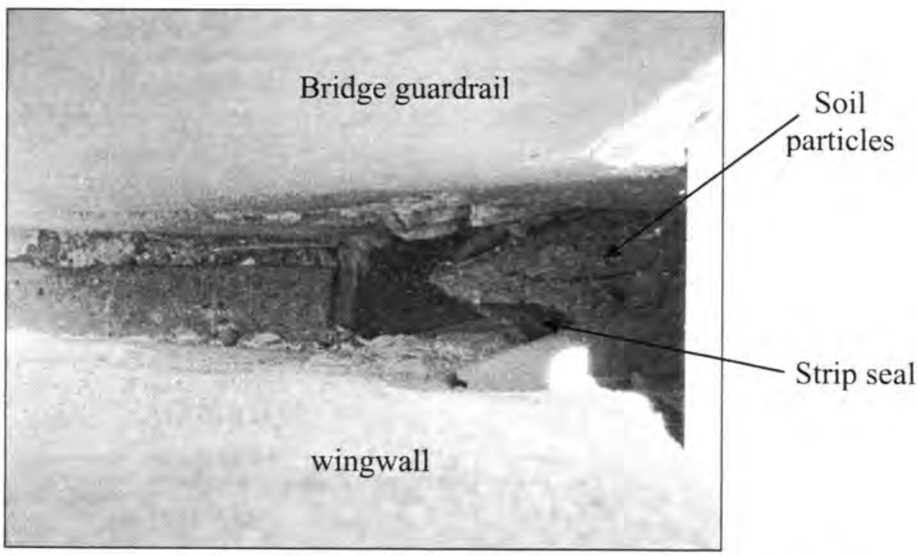


Figure 3.58 - Top View of the Strip Seal Cut Short (Bridge No. 99 - NBL)

Bridge at U.S. 20 over West Fork Little Sioux River

The bridge has steel girders. Differential settlement between the approach slab and the wingwall of 5.1 cm was observed. Although the soil of the embankment was silt, no embankment slope protection was placed under the bridge. As a result soil erosion was observed as shown in Figure 3.59.

Figure 3. 59 - Soil Erosion of the Embankment under the Bridge (Bridge at U.S. 20 over West Fork Little Sioux River)

I-29 Border Bridge (I-29 over Missouri River)

This bridge, which is maintained by South Dakota State DOT, has non-integral abutments and steel girders. Gravel was placed at the embankment. The bridge has a unique drainage system shown in Figure 3.60. The drainage system consists of a plastic container under a finger joint collecting the water and directing it through a gutter away from the bridge. This type of drainage is simple yet effective since no soil erosion was observed at this bridge. However, this system can only be implemented at non-integral abutment bridges.

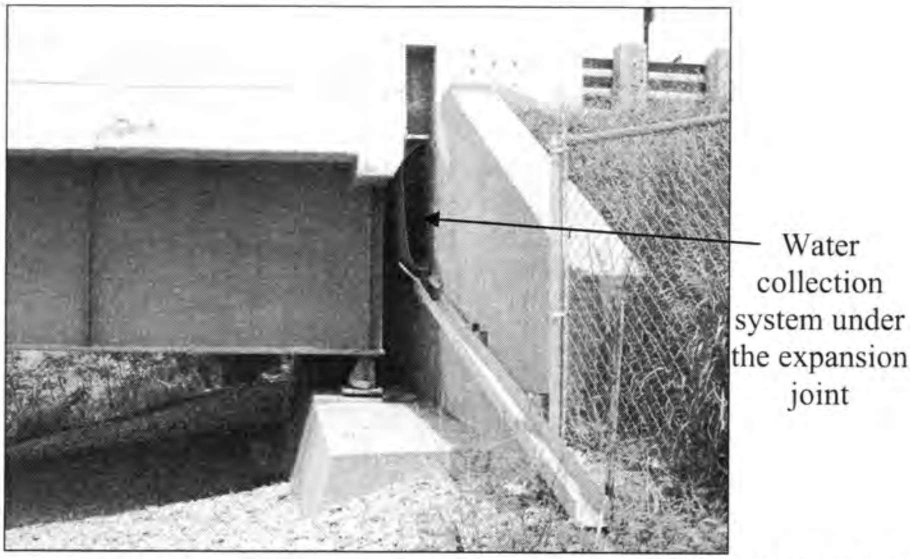


Figure 3.60 - Distinctive Drainage Design at the End of the Bridge (I-29 Border Bridge)

District 4

Bridge No. 786.3R080

This is a three span steel bridge with non-integral abutments. Differential settlement of 10.2 cm was observed at the end of the bridge. Furthermore, damage to the expansion joints was noted. Soil erosion under the abutment was observed, which led to the exposure of H piles supporting the abutment (See Figure 3.61). No slope protection was used for the bridge embankment. In addition, the approach slab panels were shaking under the impact of traffic loads indicating a void under the approach slab.

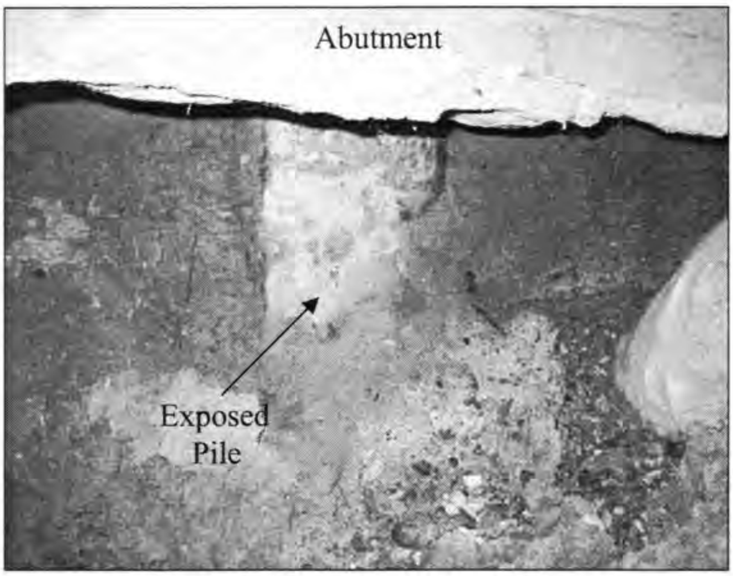
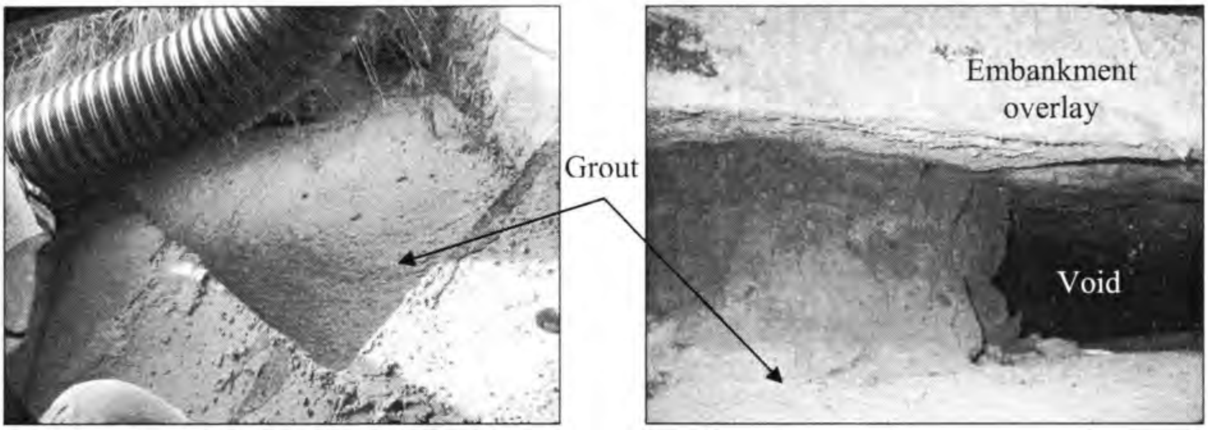


Figure 3.61 - Exposed H-pile Caused by Soil Erosion under the Abutment (Bridge No. 786.3R080)

Bridge No. 786.8R080 (I-80 over Mc Pearson Ave.)

This is a three span steel bridge with non-integral abutments. One of the observed problems was cutting the asphalt approach slab at 1.2 m intervals to provide pressure relief joints, which caused longitudinal cracks to develop as shown in Figure 3.62. Pressure relief joints were cut to allow for bridge expansion after grout was pumped around the bridge. The width of the pressure relief joint was approximately 10.2 cm. The bridge embankment has a concrete slope protection. The surface drain on the bridge deck allowed the water to fall on top of the embankment under the bridge causing erosion and saturation of the embankment soil which led to embankment settlement. As a maintenance practice, a drainage pipe was used to direct the water collected on the bridge deck away from the embankment. Furthermore, grout was pumped under the concrete slope protection to control soil erosion (See Figure 3.63). The end drain was observed blocked with soil particles (See Figure 3.64) which led to settlement and cracking of the pavement as the water collected between the subgrade and the approach slab.

Figure 3.62 - Pressure Relief Joint Made by Cutting the Approach Slab (Bridge No. 786.8R080)



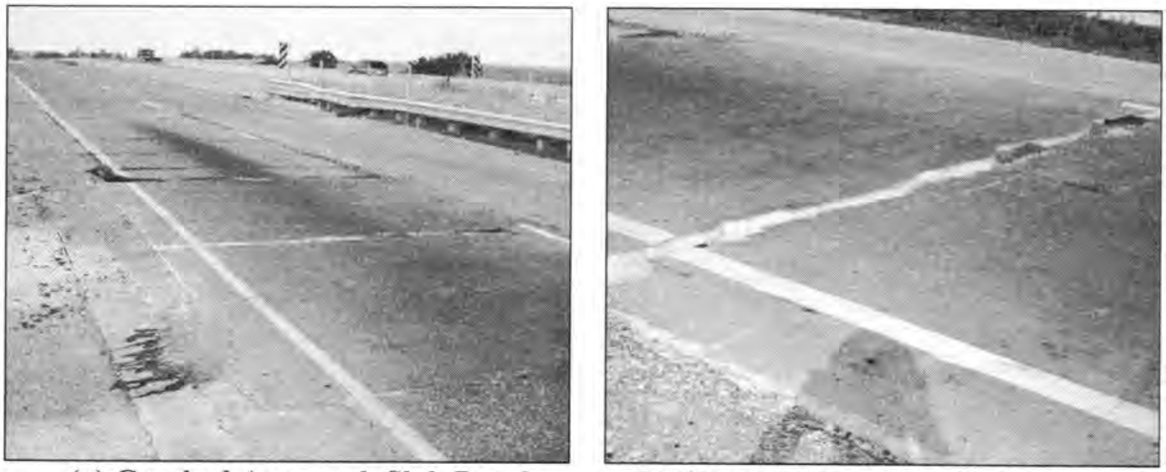
(a) Top View **(b) Side View**

Figure 3.63- Grout Pumped under the Slope Protection to Fill the Void Caused by Erosion (Bridge No. 786.8R080)

Figure 3.64 - End Drain Plugged with Soil (Bridge No. 786.8R080)

Bridge No. 788.5R080 (I-80 over Hwy 6)

This bridge, which was constructed in 1968, has four spans and non-integral abutments. Figure 3.65 shows the observed faulting of bridge approach panels. Although the approach slab section was patched 5 times with the most recent patch in spring 2003, the approach slab was wavy and cracked and differential settlement was observed. In addition, the outlet of the end drain could not be located (See Figure 3.66). Cracked concrete slope protection of the bridge embankment was observed (See Figure 3.67).



(a) Cracked Approach Slab Panels **(b) Closer Image of Faulted Approach Slab**

Figure 3.65 - Faulting of Bridge Approach Panels (Bridge No. 788.5R080)

Figure 3.66 - Drainage Outlet Could Not be Located (Bridge No. 788.5R080)

Figure 3.67 - Cracking of Concrete Slope Protection (Bridge No. 788.5R080)

Bridge No. 7821.1L680 (I-80 over County Road L34)

The bridge has three spans and non-integral abutments. At the west bound the expansion joint was poorly sealed and failure at the approach slab shoulder, which may be caused by erosion and loss of support, (See Figure 3.68) was observed. Figure 3.69 shows map and longitudinal cracks developed at the expansion joint. Differential settlement, which was measured at the approach slab shoulder, of 5.1 cm between the approach slab and the bridge deck was also noted. There was no slope protection under the bridge; therefore, soil erosion and settlement of the bridge embankment were observed.

Figure 3.68 - Failure at the Approach Slab Shoulder (Bridge No. 7821.1L680 - WBL)

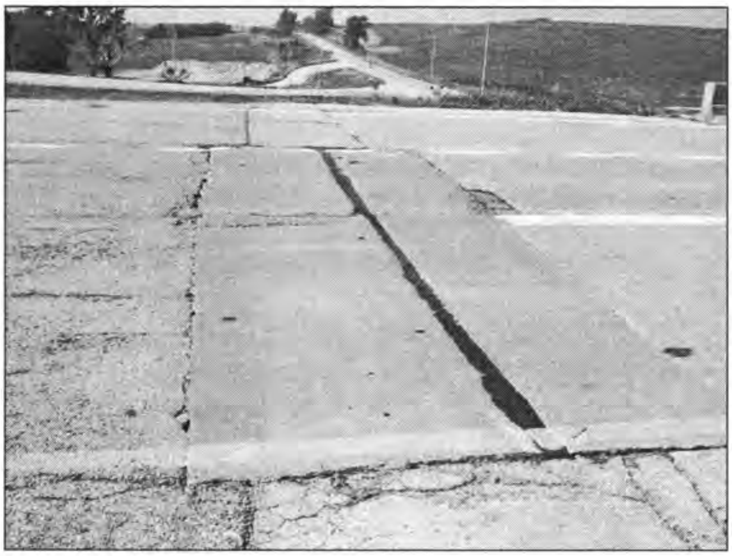
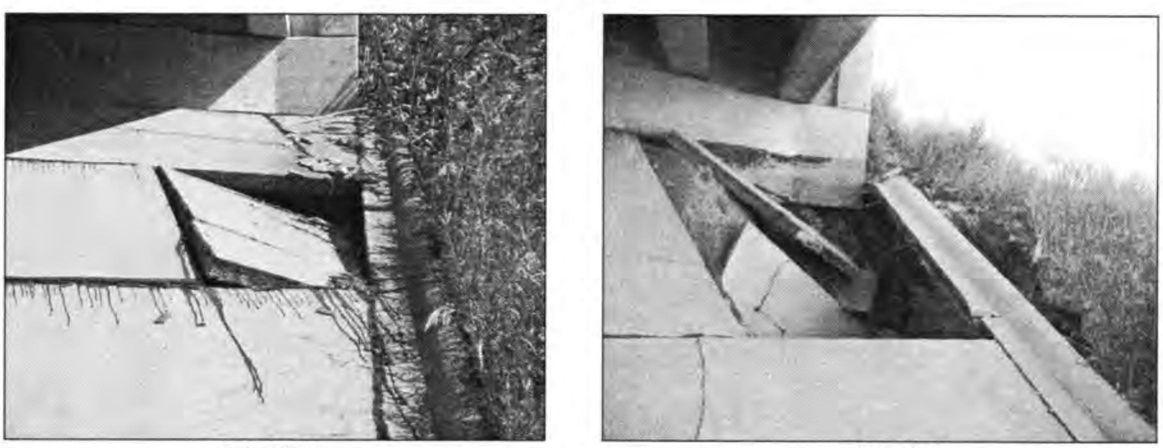


Figure 3.69 - Various Cracking at the Bridge Approach (Bridge No. 7821.1L680 - EBL)

Bridge No. 78071.5R29 (I-29 over I-680)

This bridge has integral abutments and concrete girders. Damage of the expansion joint was visible. Figure 3.70 shows the bridge surface drain at the north bound lane. This figure illustrates that no drainage pipe was used to direct the water collected on the bridge surface away from the bridge embankment causing soil erosion under the concrete slope protection. Excessive erosion led to the failure of the slope protection at both bounds and exposure of the H-piles supporting the abutments (See Figures 3.71 and 3.72). Settlement of the bridge embankment at the south bound lane was 26.7 cm (See Figure 3.73). Water was observed at the bottom of the bridge embankment which may affect the subgrade of the road under the bridge (See Figure 3.74).

Figure 3.70 - Surface Drain of the Bridge Deck Allowing Water to fall on the Embankment (Bridge No. 78071.5R29)



(a) NBL **(b) SBL**

Figure 3.71 - Failure of Concrete Slope Protection (Bridge No. 78071.5R29)

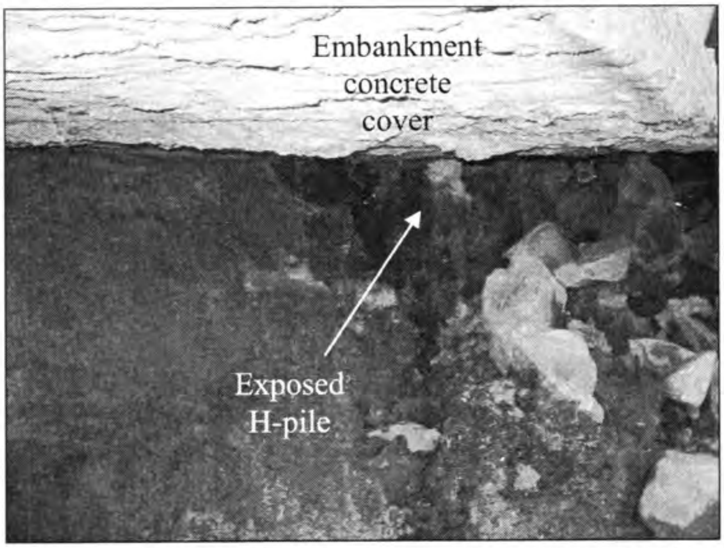


Figure 3.72 - Exposed H-pile (Bridge No. 78071.5R29)

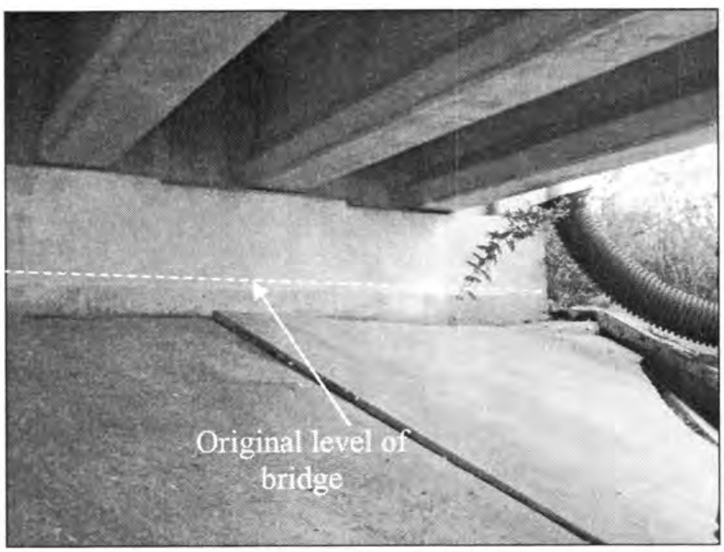


Figure 3.73 - Settlement of the Bridge Embankment (Bridge No. 78071.5R29)

Figure 3.74 - Water Observed at the Bottom of the Embankment (Bridge No. 78071.5R29)

District 5

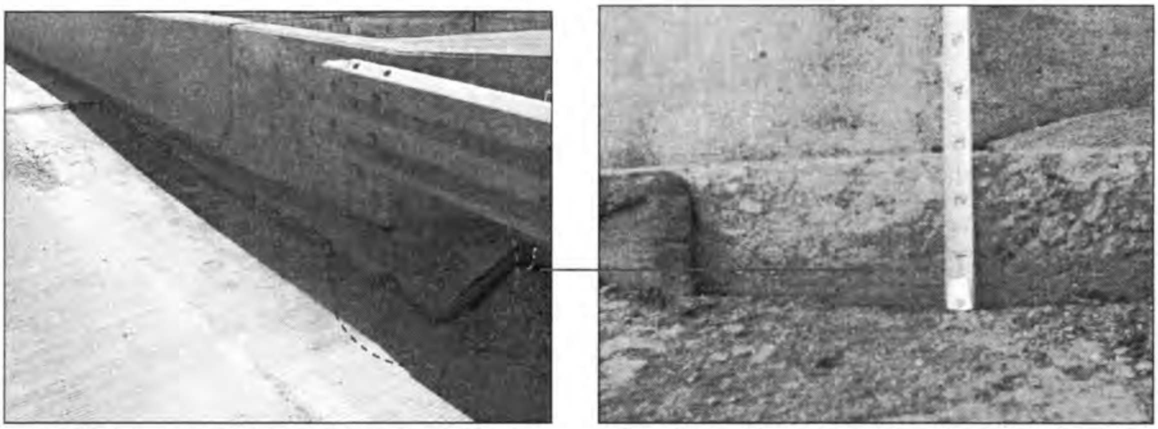
Bridge at Clay St. Crossing I-35

This is a two span concrete bridge with integral abutments. Gravel was placed at the bridge embankment for slope protection.

This bridge does not have a subdrain behind the abutment and the bridge end drains were constructed away from the bridge increasing erosion as more water accumulated on the bridge.

At the west end, differential settlement of 7.6 cm relative to the wingwall was observed at the bridge approach (See Figures 3.75 and 3.76). The width of the expansion joint at the west end was 14 cm. Parts of the joint filler material were missing (See Figure 3.77) allowing water to flow behind the bridge abutment which increased the soil erosion around the bridge. As a result, a void developed under the approach slab which was 28 cm deep at the abutment and extended to 1.8 m away from the abutment was observed (See Figure 3.78). The end drain at the west end was observed to be in good working condition.

Figure 3. 75 - Settlement of the Approach Slab (Bridge at Clay St. Crossing I-35)



(a) Settlement of the Approach Slab | (b) Closer Image of the Approach Slab Settlement

Figure 3.76 - Settlement of Approach Slab Relative to Wingwall (Bridge at Clay St. Crossing I-35)

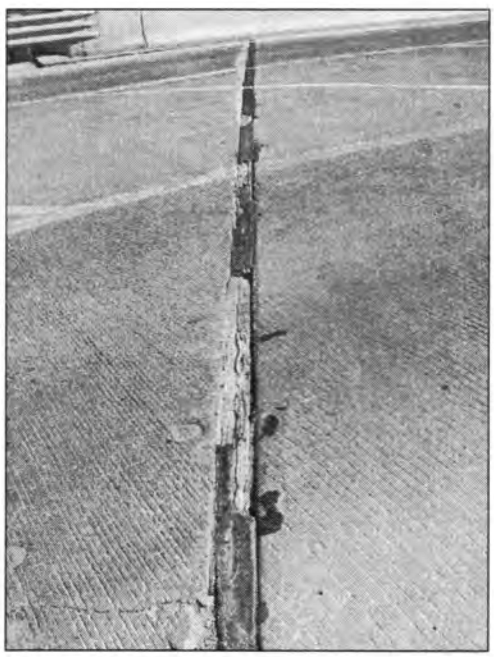


Figure 3.77 - Deteriorated Expansion Joint Sealer (Bridge at Clay St. Crossing I-35)

Figure 3.78 - Void Developed under Approach Slab (Bridge at Clay St. Crossing I-35)

At the east end, differential settlement, which was measured at the wingwall, between the approach slab and the bridge deck was 2.5 cm (See Figure 3.79), and the expansion joint width was 12.7 cm. The sealer deteriorated shortly after construction, and the joint filler was missing leaving a large gap (See Figure 3.80) where water can flow and erode the backfill under the approach slab. The void developed under the approach slab was approximately 10.2 cm deep (See Figure 3.81). Another indication of poor water management was the erosion along the abutment sides (See Figure 3.82). Furthermore, concrete spalling was observed at the expansion joint.

Figure 3.79 - Differential Settlement at the Bridge Approach (Bridge at Clay St. Crossing I-35 - East End)

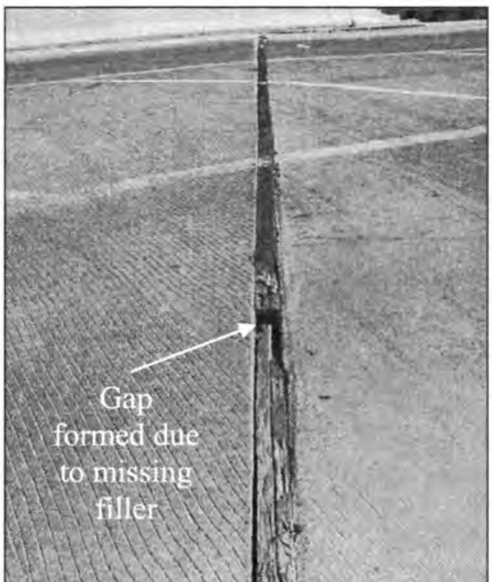


Figure 3.80 - Missing and Deteriorated Filler Material at the Expansion Joint (Bridge at Clay St. Crossing I-35 - East End)

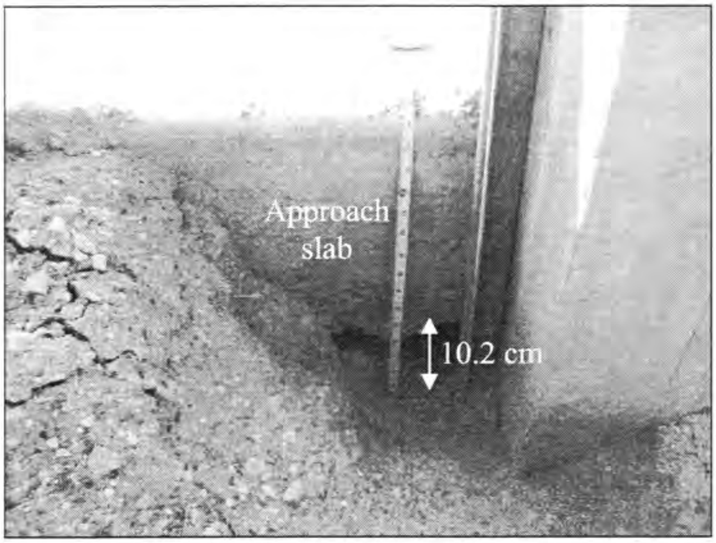


Figure 3.81 - Void Developed under the Approach Slab (Bridge at Clay St. Crossing I-35 - East End)

Figure 3.82 - Erosion Along the Abutment Side (Bridge at Clay St. Crossing I-35 - East End)

Bridge at Old U.S. 218 Crossing Henry St.

This is a three span bridge with integral abutments. The approach slabs were replaced in May 2004 due to the failure of the paving notch. After replacing the approaches, the new expansion joints were 6.4 cm wide. However; differential settlement between the bridge deck and the approach slab was observed (See Figure 3.83). At the south end of the bridge, the abutment moved laterally a distance of 2.5 cm away from the bridge embankment (See Figure 3.84), while the embankment settled 7.6 cm. At the north end of the bridge, concrete slope protection panels were removed for replacement due to uneven settlement, cracking, and failure. The soil used at the bridge embankment was silty clay (See Figure 3.85). Rocks were used at the sides of the abutment to minimize erosion. Similar to other bridges inspected in district 5, this bridge does not have a subdrain around the abutment.

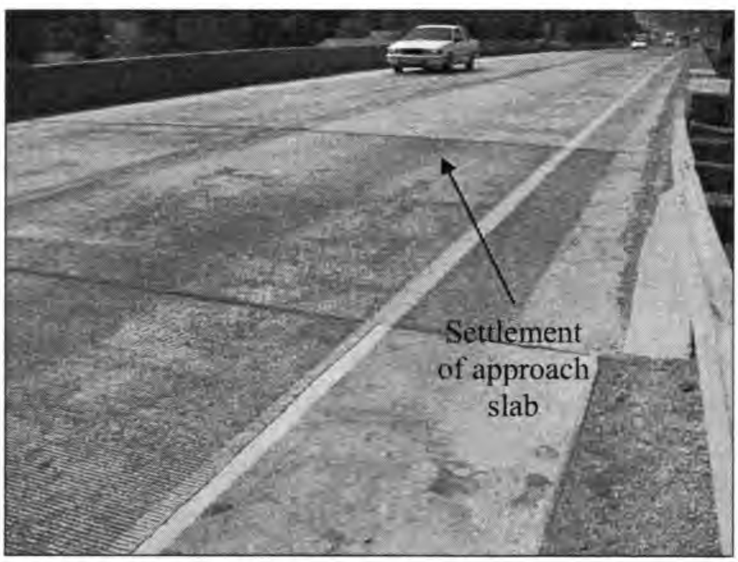


Figure 3.83 - Differential Settlement at Recently Replaced Bridge Approach (Bridge at Old U.S. 218 Crossing Henry St.)

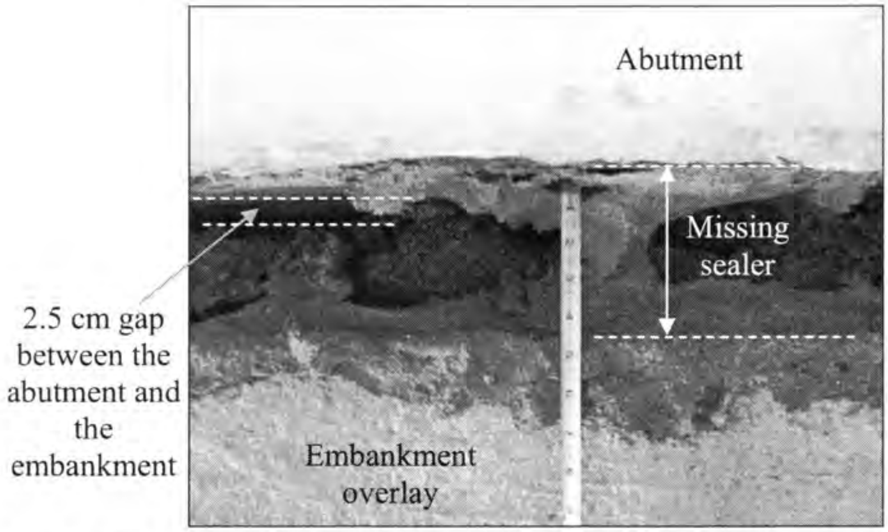


Figure 3.84 - Lateral Movement of the Abutment (Bridge at Old U.S. 218 Crossing Henry St.)

Figure 3.85 - Bridge Embankment Prior to Placing the New Overlay (Bridge at Old U.S. 218 Crossing Henry St.)

<u>Bridge at U.S. 218 over South Skunk River</u>

This is a five span bridge with concrete girders at the south bound lane and steel girders at the north bound lane. The bridge has non-integral abutments at both bounds. Glacial till soil was used to build the embankment with no slope protection under the bridge.

At the south end of the south bound lane, the approach slab was not yet constructed at the time of inspection. It was observed that the strip seal at the expansion joint was cut short which allowed water to flow downward to bridge embankment (See Figure 3.86). Wet drains were used during embankment construction to allow for faster consolidation of the foundation soil. These wet drains were observed filled with soil (See Figure 3.87). At the north end of the north bound lane erosion of the bridge embankment was noted (See Figure 3.88).

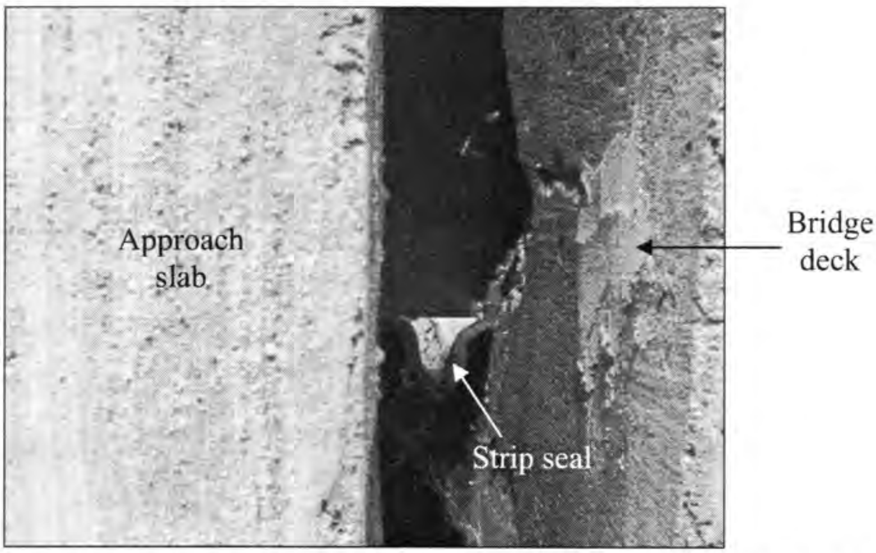


Figure 3. 86 - Side View of the Strip Seal which was Cut Short (Bridge at U.S. 218 over South Skunk River - South End of SBL)

Figure 3.87 - "Wet-Drain" Filled with Soil Particles (Bridge at U.S. 218 over South Skunk River - South End of SBL)

Figure 3.88 - Erosion of the Bridge Embankment (Bridge at U.S. 218 over South Skunk River - North End of NBL)

<u>Bridge at U.S. 218 (Near Exit 42)</u>

This is a two span bridge with steel girders and integral abutments. Gravel was used as slope protection for the bridge embankment. At the north end of the north bound lane, a void under the approach slab that ranged from 7.6 cm to 20.3 cm was developed which lead to faulting of the approach slab panels. This void was grouted and the approach slab panels were replaced. After replacement, the expansion joint was 7.6 cm wide and flexible foam was used as joint filler. The joint was poorly sealed as shown in Figure 3.89, and concrete spalling was still noticeable. No surface drain on the bridge was constructed; however, water was directed away from the bridge by sloping the shoulders away from the bridge which caused erosion along the abutment sides. At the south end of the north bound lane, differential settlement was 7.6 cm, which was measured at the wingwall. A 10.2 cm void under the approach slab was grouted as shown in Figure 3.90. Erosion along the abutment sides was also noted.

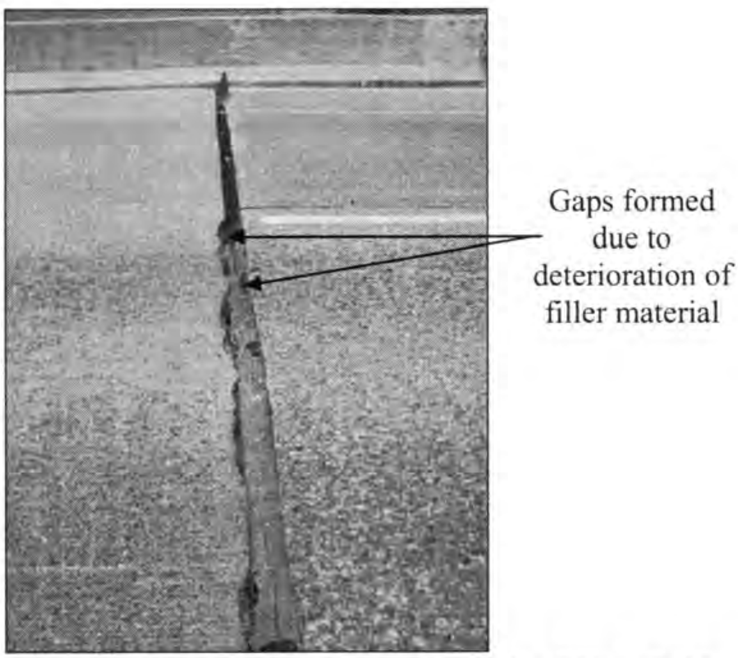


Figure 3.89 - Deteriorated Flexible Foam which was used as Joint Filler (Bridge at U.S. 218 - North End of NBL)

Figure 3.90 - Grouting Under the Approach Slab (Bridge at U.S. 218 - South End of NBL)

Bridge No. 9266.2R (over Creek)

This is a three span bridge with concrete girders and integral abutments. The bridge which was resurfaced using an asphalt overlay at the west end of the west bound, was significantly deteriorated and cracked as shown in Figures 3.91 and 3.92. The width of the expansion joint was 10.2 cm with flexible foam used as joint filler. Differential settlement between the approach slab and the bridge deck was 2.5 cm measured at the approach slab shoulder. Grout was pumped under the approach slab due to void development. Glacial till was used as embankment material with no slope protection. Erosion between the embankment and abutment backwall was noticeable (See Figure 3.93). The subdrain outlet was blocked with soil particles with no sign of water draining out (See Figure 3.94).

Figure 3.91 - Poorly Resurfaced Approach Slab (Bridge No. 9266.2R - West End of WBL)

Figure 3.92 - Transverse Cracking of the Asphalt Overlay (Bridge No. 9266.2R - West End of WBL)

Figure 3.93 - Erosion of the Embankment Soil (Bridge No. 9266.2R - West End of WBL)

Figure 3.94 - Subdrain Outlet Blocked with Soil Particles (Bridge No. 9266.2R - West Eend of WBL)

<u>Bridge No. 9265.6L (over Creek)</u>

This is a three span bridge with integral abutments. At the east end of the bridge, differential settlement between the bridge deck and the approach slab was 2.5 cm measured at the wingwall. The width of the expansion joint was 11.4 cm. Parts of the flexible foam joint filler completely came out exposing the paving notch (See Figure 3.95), which was covered with approximately 7.5 cm of soil. Grout was pumped under the approach slab due to void development behind the abutment. Furthermore, erosion between the bridge embankment and the backwall was observed (See Figure 3.96a) and rocks were used to minimize erosion (See Figure 3.96b).

Figure 3.95 - Missing Filler Material Exposing the Pavement Notch (Bridge No. 9265.6L - East End of WBL)

(a) Erosion at the Bridge Embankment

(b) Rocks used to Control Erosion

Figure 3.96 - Erosion between the Bridge Embankment and the Abutment, and Rocks Placed to Reduce Erosion (Bridge No. 9265.6L - East End of WBL)

District 6

Bridge No. 574.2S001 (Hwy 1 over Cedar River)

This bridge, which was constructed in 1992, has concrete girders and non-integral abutments. Differential settlement of 7.6 cm was measured at the bridge approach wingwall (See Figures 3.97). The strip seal of the expansion joint was cut short and filled with soil. The embankment under the bridge was built using loess with no slope protection (See Figure 3.98). Water was observed flowing through the expansion joint to the bridge embankment. To fill the void developed behind the abutment, the bridge was grouted in 2003.

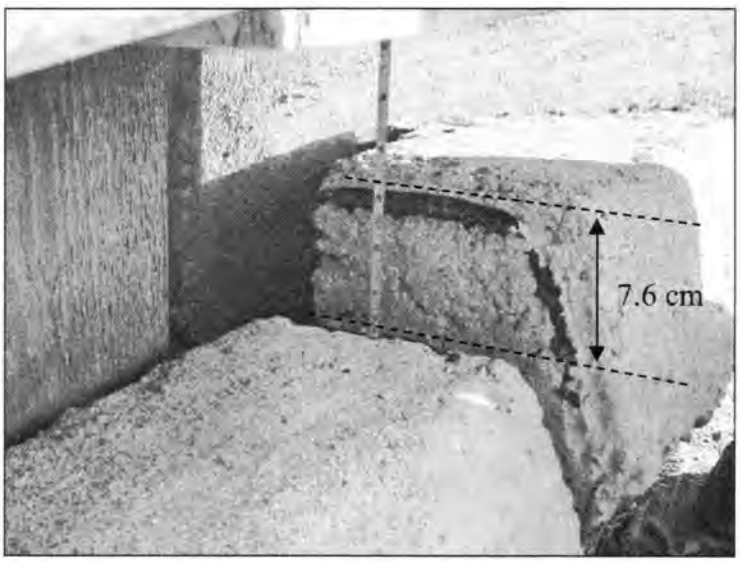


Figure 3.97 - Differential Settlement at the Bridge Approach (Bridge No. 574.2S001 - SBL)

Figure 3.98 - Wet Loess Soil Observed at the Bridge Embankment (Bridge No. 574.2S001)

Bridge No. 5289.8R218 (Hwy 218 over Hwy 921)

This bridge, which was constructed in 1983, has three spans and integral abutments. Although maintenance reports indicated that grouting, which seeped around the bridge (See Figure 3.99 and 3.100), was performed twice to fill the void developed under all approach slabs, additional settlement and loss of material created a void under the approach slab. This void caused vibration of the approach slab under the impact of traffic loads. The approach slab shoulder was drilled during inspection to measure the depth of the void. The three drilled holes indicated 5.1 cm gap under the bridge approach. Figure 3.101 shows the distance between the approach slab surface and the base material, which was 35.6 cm with a slab thickness of 30.5 cm. The void developed resulted in 5.1 cm. differential settlement between the bridge deck and the bridge approach (See Figure 3.102) and significant cracks and faulting of the approach slab (See Figure 3.103). Flexible foam was used as expansion joint filler which was not sealing the joint as shown in Figure 3.104. A void under the concrete slope protection (See Figure 3.105) and uneven settlement and cracking of the concrete slope protection panels covering the bridge embankment were observed (See Figure 3.106). Furthermore, the bridge embankment settled 11.4 cm. The lateral movement of the abutment relative to the bridge embankment, caused by bridge expansion, resulted in a 3.8 cm wide and 1.4 m deep void (See Figure 3.107).

Figure 3.99 - Grout Seeping from under the Concrete Cover (Bridge No. 574.2S001)

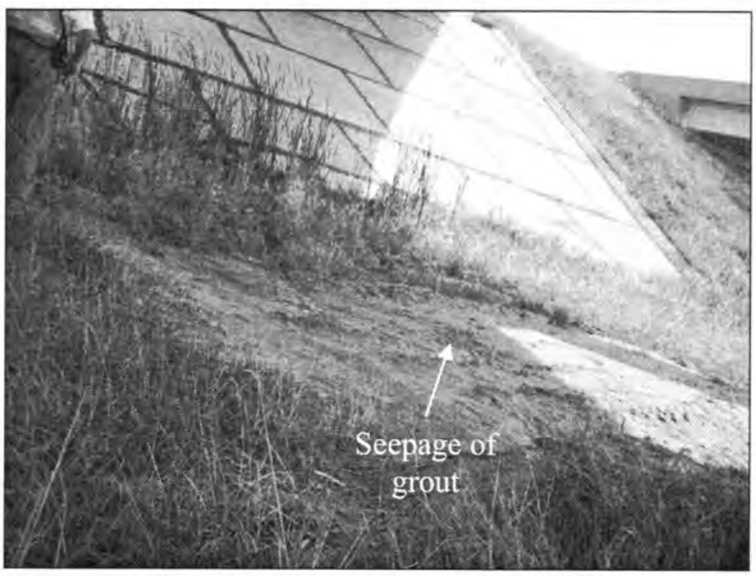
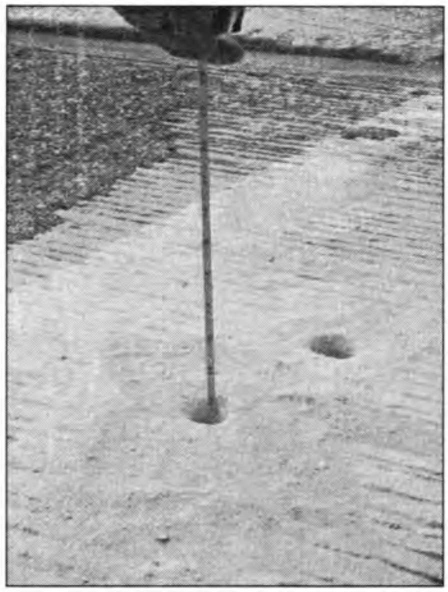


Figure 3.100 - Grout Seeping from the Bottom of the Embankment (Bridge No. 574.2S001)

(a) Drilling at the Approach Slab



(b) Measuring the Void

Figure 3.101 - Drilling at the Approach Slab Shoulder for Void Detection (Bridge No. 574.2S001)

Figure 3.102 - Differential Settlement at the Approach Slab (Bridge No. 574.2S001)

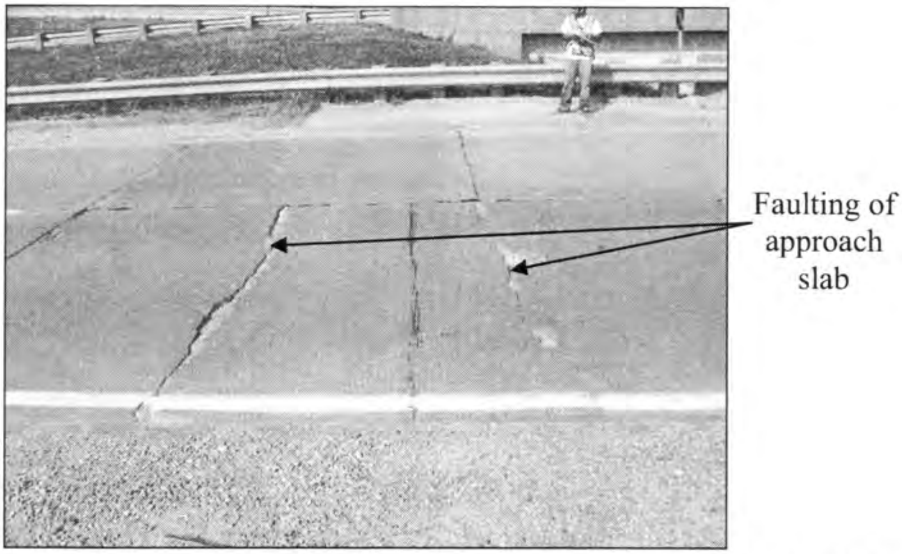


Figure 3.103 - Faulting of Approach Slab Concrete Panels (Bridge No. 574.2S001)

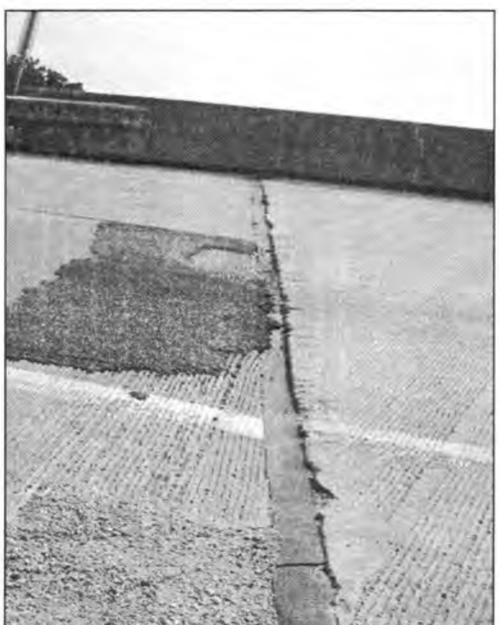
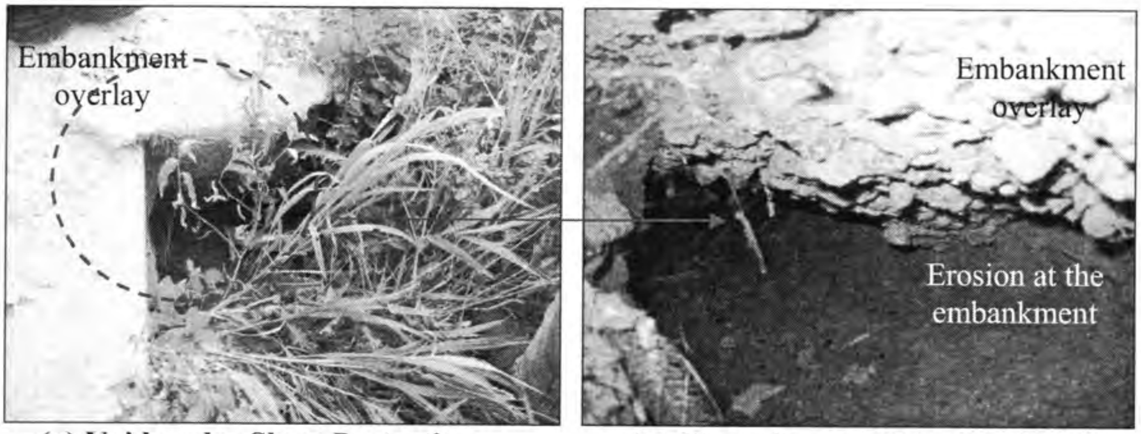


Figure 3.104 - Flexible Foam used as Joint Filler (Bridge No. 574.2S001)



(a) Void under Slope Protection **(b) Closer Image of the Developed Void**

Figure 3.105 - Void Created by Erosion and Settlement after Grouting (Bridge No. 574.2S001)

Figure 3.106 - Cracking of the Slope Protection (Bridge No. 574.2S001)

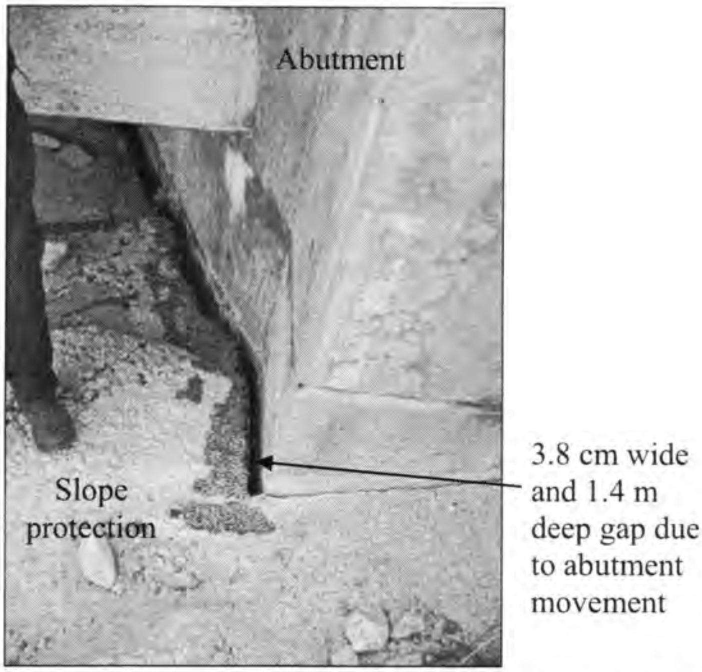


Figure 3.107 - Lateral Movement of the Abutment Away from the Bridge Embankment (Bridge No. 574.2S001)

Bridge No. 5200.5R380 (I-380 over Clear Creek)

This bridge, which was constructed in 1971, has three spans and integral abutments. Cracks and concrete spalling were observed around the expansion joint (See Figure 3.108). Soil erosion and ponding of water were noted on the bridge embankment (See Figures 3.109 and 3.110). Rocks and pillows with special concrete mix were used for slope stabilization, as shown in Figure 3.111. According to the maintenance report, the near end of the right lane of the bridge approach was grouted using 3.8 m^3 of flowable mortar in fall 2002.

Figure 3.108 - Concrete Spalling and Cracking at the Expansion Joint (Bridge No. 5200.5R380)

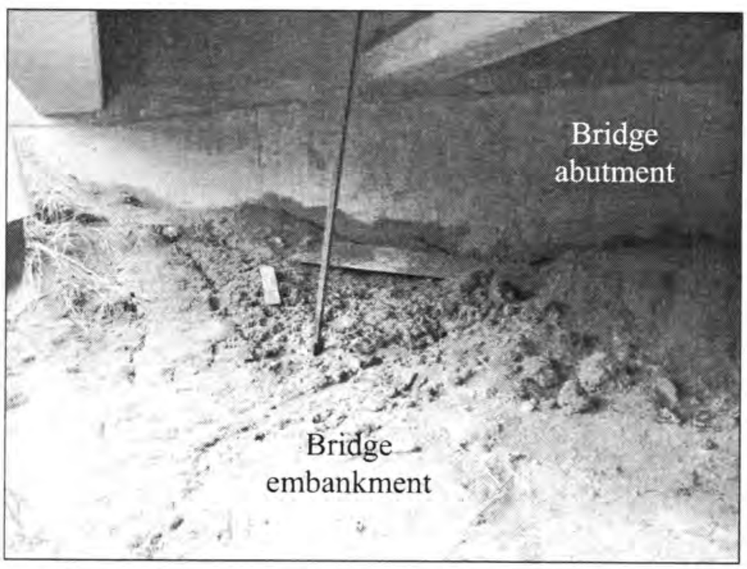


Figure 3.109 - Soil Erosion of the Bridge Embankment (Bridge No. 5200.5R380)

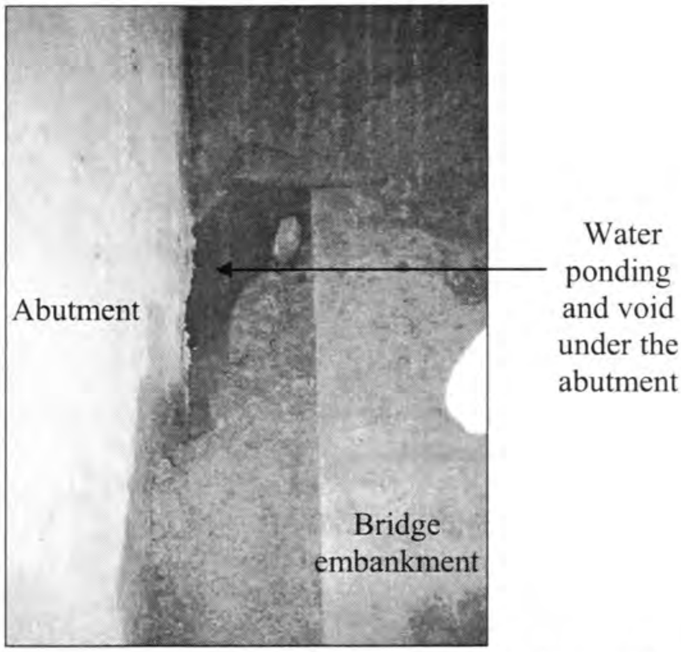


Figure 3.110 - Water Ponding at the Embankment (Bridge No. 5200.5R380)

Figure 3.111 - Rocks and Pillows used for Embankment Stabilization (Bridge No. 5200.5R380)

Bridge No. 5200.8R380 (I-380 over U.S. 6)

This bridge, which was constructed in 1970, has three spans with steel girders and non-integral abutments. The concrete slope protections of the embankments at both ends of the bridge were significantly damaged as shown in Figure 3.112 due to erosion. Furthermore, the embankment under the bridge experienced 12.7 cm settlement. Lateral movement of the abutment away from the embankment due to bridge expansion was 5.1 cm. No differential settlement was observed between the bridge deck and the bridge approach, and the expansion joints were in a satisfactory condition. Maintenance report for this bridge indicated that flowable mortar was used to fill a void developed at the far end of the left lane in fall 2002. Flowable mortar was used again in July 2003 to fill the additional void developed at the same approach slab.

Figure 3.112 - Severe Damage of Concrete Slope Protection Due to Erosion (Bridge No. 5200.8R380)

Bridge No. 5200.0R380 (I-380 over I-80)

This bridge was not inspected during the field visit; however, some figures and maintenance reports were provided by Iowa DOT. This bridge has four spans and non-integral abutments. Figure 3.113 shows uneven settlement of the concrete slope protection of the bridge embankment. The near end of the right lane of the bridge was grouted using flowable mortar in August 2003 to fill the void developed behind the abutment exposing the H-piles (See Figures 3.114 to 3.117).

Figure 3.113 - Uneven Settlement of Slope Protection Panels (Bridge No. 5200.0R380)

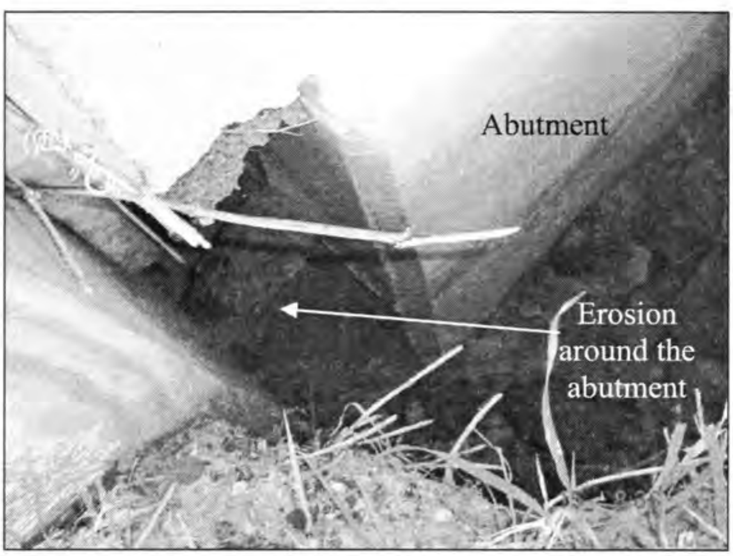


Figure 3.114 - Soil Erosion around the Abutment (Bridge No. 5200.0R380)

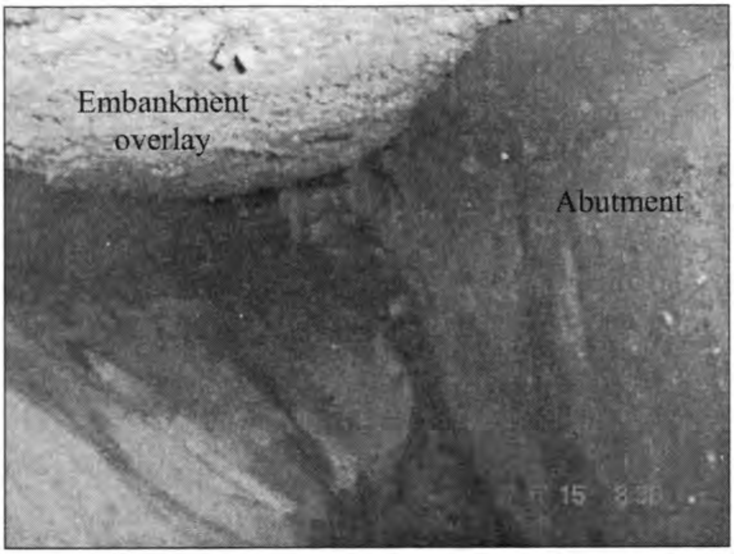


Figure 3.115 - Erosion under the Concrete Slope Protection (Bridge No. 5200.0R380)

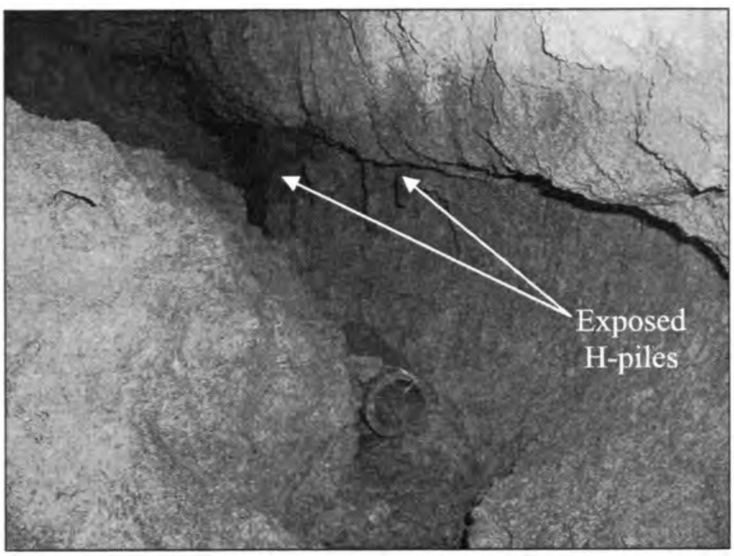


Figure 3.116 - Void Created under Bridge Approach Exposing H-Piles (Bridge No. 5200.0R380)

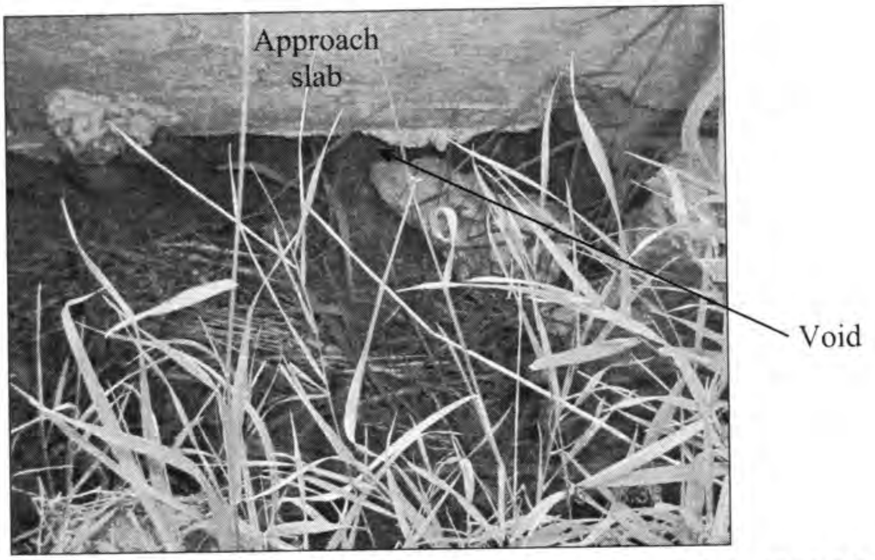


Figure 3.117 - Void Developed under the Approach Slab (Bridge No. 5200.0R380)

3.2.2 Bridges under Construction

Eight under construction bridges were inspected. Practices that do not match Iowa DOT specifications were observed at these sites. Although, Iowa DOT specifications require using porous backfill material around the subdrain and compaction of granular backfill every 200 mm (8 in.), no compaction of the granular backfill was observed at five bridges and no porous backfill around the subdrain was used at six bridges. Furthermore, it was noticed that Iowa DOT does not specify a range of moisture content for granular backfill and in the field the backfill material was placed at moisture contents within the range of bulking moisture. At these moisture contents the tensile forces between water and soil particles are large and the specified compaction can not be achieved. Other observed construction problems at these sites included: (1) subdrain behind the bridge abutment filled with soil particles at four bridge sites; and (2) poor construction of the paving notch. Table 3.2 summarizes the observations at all inspected under construction bridges, and the tests performed on the backfill materials used at these sites.

Table 3.2 - Summary of Major Problems and Tests Conducted at Bridges under Construction

Bridge location	District	Major problems	Tests conducted
35th St. Bridge over I-235	1	• No compaction of backfill • Backfill at bulking moisture content • Subdrain filled with soil	Grain size distribution, moisture content, and relative density
Polk blvd. Bridge	1	• No compaction of backfill • Backfill at bulking moisture content • No porous fill around subdrain • Subdrain filled with soil	Grain size distribution, moisture content, relative density, DCP, and nuclear gauge
19th St. Bridge over I-235	1	None	Grain size distribution, and relative density
Pennsylvania Ave Bridge	1	• Subdrain filled with soil	None
E 12th St. Bridge	1	• No compaction of backfill	Grain size distribution, and relative density
Euclid Ave. Bridge	1	• No compaction of backfill • Subdrain filled with soil	None
Bridge over Union Pacific	3	• No compaction of backfill • Backfill at bulking moisture content • No porous fill around subdrain • Poor construction of paving notch	Grain size distribution, and air permeability test
57.6R030	6	• Poorly constructed paving notch	Grain size distribution

35th Street Bridge over I-235 (District 1)

This is a two span bridge with integral abutments. Concrete slope protection was used for the bridge embankment. At the north end of the north bound lane, the abutment was not yet constructed when this bridge was inspected; however, the center pier and the girders up to the center pier were completed for the north bound lane (See Figure 3.118). Granular backfill was used behind the backwall and the abutment (See Figure 3.119). At the south end of the north bound lane, porous fill was placed around the subdrain as shown in Figure 3.120. The outlet of the subdrain shown in Figure 3.121 was observed filled with soil particles. Figure 3.122 shows the subdrain at the south end buried along the side of the back wall and under the embankment overlay. Air Permeability Test (APT) shown in Figure 3.123 (White *et al.* 2004 for more details about this test) was performed on the porous backfill material at 3 locations behind the abutment. The average permeability coefficient was 28.4 cm/s. A samples of backfill materials used at the site were tested in the laboratory.

Table 3.3 summarizes the results of the laboratory tests performed on backfill materials. Figures 3.124 and 3.125 show the grain size distribution of granular and porous backfill materials used at the north and south ends, respectively. According to the Unified Soil Classification System (USCS), the granular backfill was classified as poorly graded sand (SP); while the porous backfill was classified as poorly graded gravel (GP). Vibrating table tests (ASTM D4253-00) were performed on the granular backfill material at several moisture contents to evaluate the minimum and maximum densities, material compactibility, and the bulking moisture content range (See Figure 3.126). For the porous backfill, moisture content could be determined. Compactibilities of 1.052 and 0.157 were calculated for the granular and the porous backfill materials respectively, which indicate high compactibility for the granular backfill and low compactibility for the porous backfill (See Hilf 1991). The minimum and maximum dry densities were 14.3 kN/m^3 and 18.9 kN/m^3, respectively for the granular backfill and 14.8 kN/m^3 and 15.7 kN/m^3 for porous backfill. The natural moisture content for the granular and porous backfill materials were 3.9% and 4.2%, respectively. The natural moisture content of the granular backfill was within the bulking moisture content range (≈ 3% to 7%) where compaction requirements could not be achieved.

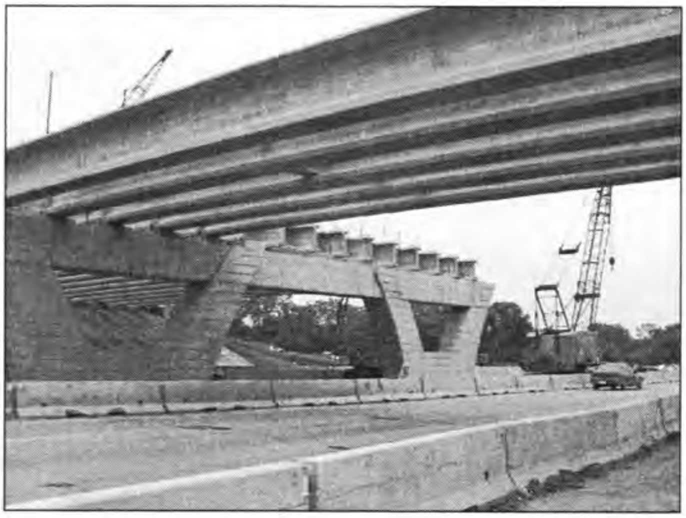


Figure 3.118 - Girders under Construction (35th Street over I-235 - NBL)

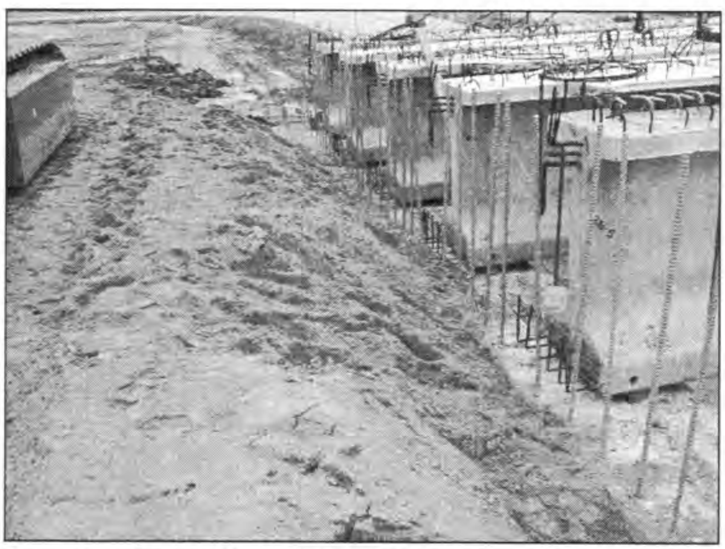


Figure 3.119 - Granular Backfill used behind the Abutment (35th Street over I-235)

Figure 3.120 - Porous Fill Placed around the Subdrain (35th Street over I-235)

Figure 3.121 - Subdrain Outlet at the Bottom of the Embankment (35th Street over I-235)

Figure 3.122 - Subdrain Placed Along the Side of the Abutment (35th Street over I-235)

Figure 3.123 - Air Permeability Test Performed at Porous Backfill (35th Street over I-235)

Table 3.3 - Properties of Backfill Material (SBL)

	Granular backfill	Porous backfill
Classification	SP	GP
Natural moisture content	3.9%	4.2%
Bulking moisture content	3-7 %	-
Compactibility, F	1.052	0.157
Maximum dry density (kN/m^3)	18.9	15.7
Minimum dry density (kN/m^3)	14.3	14.8

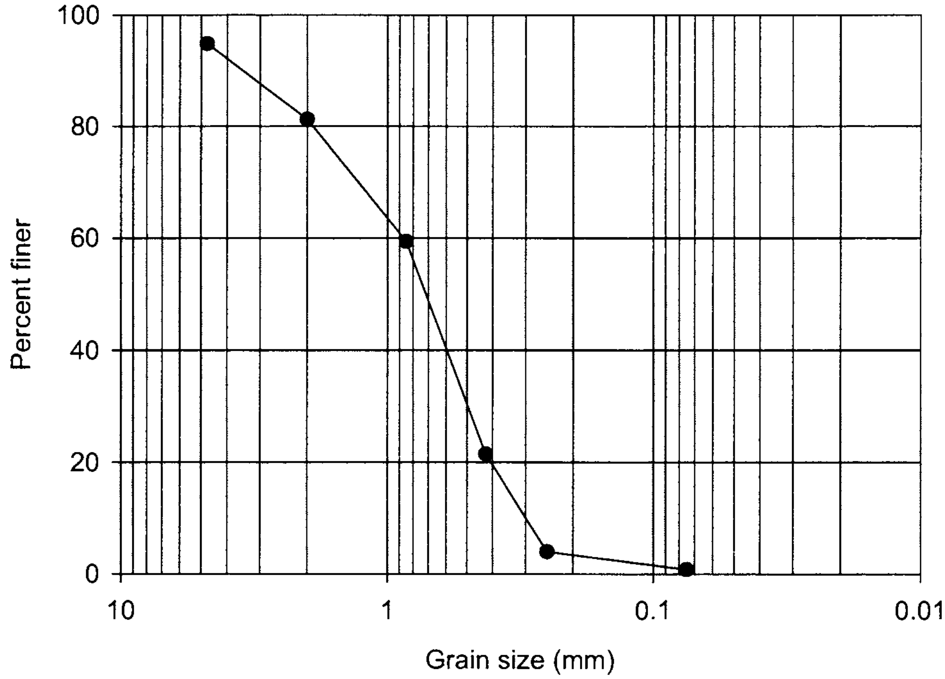


Figure 3.124 - Gradation of Granular Backfill Material used at the North End; Classified as SP (35th Street over I-235)

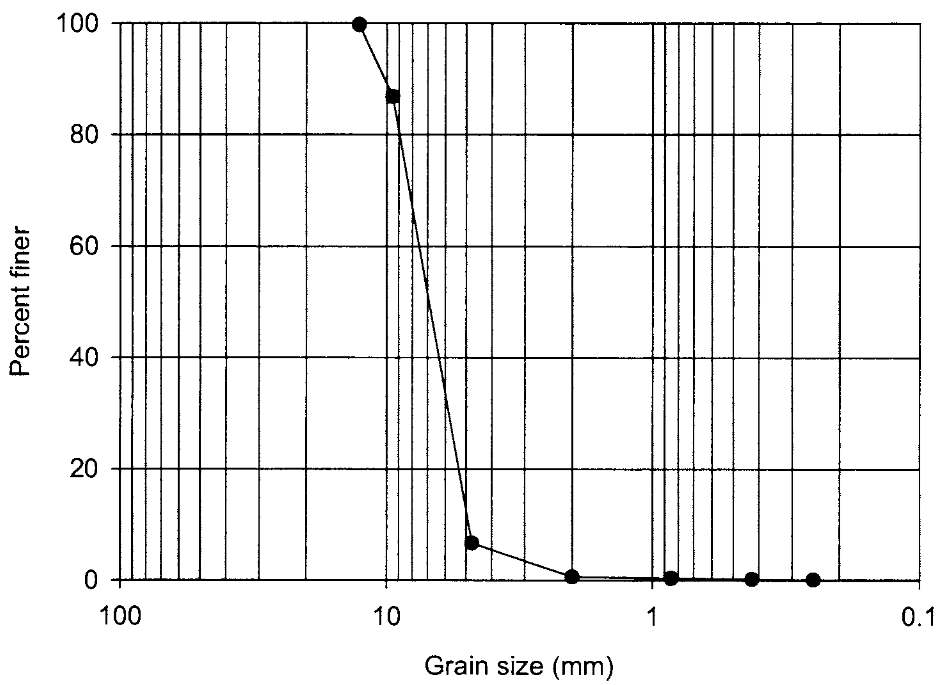


Figure 3.125 - Gradation of Porous Backfill Material used at the South End; Classified as GP (35th Street over I-235)

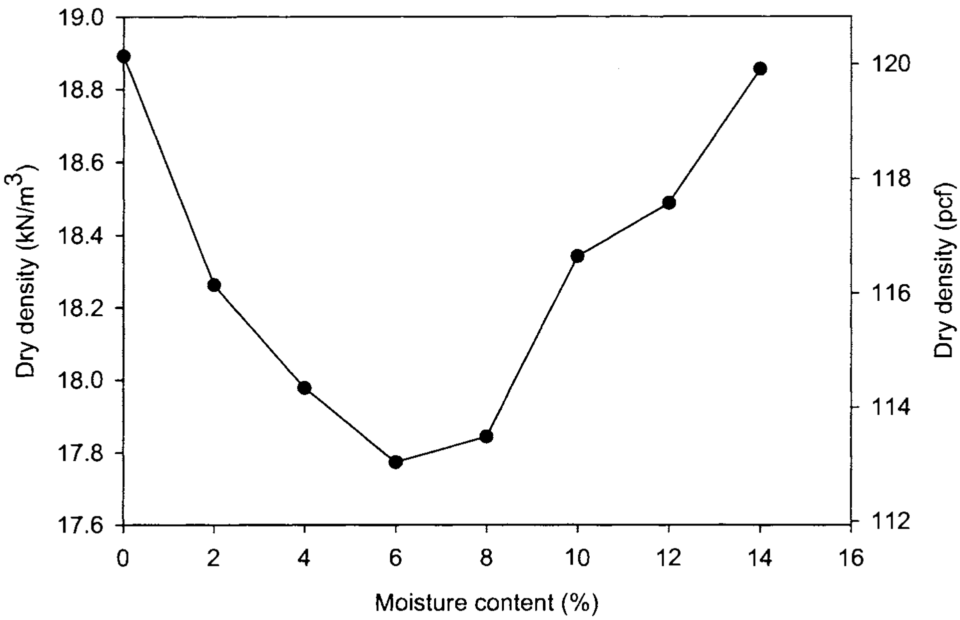


Figure 3.126 - Dry Density–Moisture Content Relationship of Granular Backfill used at the North End (35th Street over I-235)

Polk Blvd Bridge over I-235 (District 1)

This bridge was inspected four times- twice in June, once in July and once in October of 2003. At this location the old bridge was being replaced and widened. The new bridge has integral abutments. When inspected for the first time, the old bridge at the north bound lane was removed, the center pier of the new lane was constructed (See Figure 3.127), and the sheet piles were being installed (See Figure 3.128). The old south bound lane, which was still open for traffic had non-integral abutments and supported on a deep slab. At the south bound lane, concrete spalling of the abutment was observed (See Figure 3.129). This bound had a concrete slope protection cover for the embankment under the bridge.

The bridge was inspected again in June 2003. During this field visit the H-Piles were being installed at the north end of the north bound lane as shown in Figure 3.130. In July 2003, the bridge was visited again where retaining wall construction was taking place at the south end of the north bound lane (See Figure 3.131). At the north end, the backwall was already constructed. Granular backfill material was used behind the abutment as shown in Figure 3.132. No porous backfill was observed around the subdrain (See Figure 3.133). When examined, the subdrain was filled with sand particles.

The bridge was inspected again in October 2003. The new north bound lane was completed and opened for traffic, while the old south bound lane was removed and construction of the new bridge was in progress. During this field visit, H-piles were being driven at the north end as shown in Figure 3.134. At the south end, construction and compaction of the backfill behind the retaining wall using a vibratory base plate was taking place (See Figures 3.135 and 3.136) and the piles were already installed. The backfill material used behind the abutment was tested in the field and in the laboratory.

Dynamic Cone Penetration Test (DCP) was conducted at three locations behind the backwall at the north end of the north bound lane. Using DCP data, estimated CBR ranged from one to eight at all tested locations (See Figures 3.137 and 3.138) indicating a soft and loose backfill material. Nuclear gage test was also conducted at two locations behind the north end backwall. The average measured dry density was 16.5 kN/m^3 and the moisture content was 4.7%.

Figure 3.139 shows the grain size distribution of the granular backfill material used behind the abutments, which was classified as SP according to the USCS. Minimum and maximum density, compactibility, and bulking moisture content for granular backfill were estimated using the vibrating table test (ASTM D4253-00). The maximum dry density was 17.8 kN/m^3, the minimum dry density was 15.1 kN/m^3 and the bulking moisture content ranged from 3% to 5% (See Figure 3.140). The measured moisture content of the backfill material at the site was within this bulking moisture content range. Using the measured dry density in the field, the calculated relative density was 56% which classify the compacted backfill as medium dense material (Das 1998). Compactibility of the granular backfill of 0.54 was calculated which indicate a low compactibility for the backfill material.

Figure 3.127 - Construction of the New North Bound Lane Bridge (Polk Blvd. Bridge)

Figure 3.128 - Installation of the Sheet Piles (Polk Blvd. Bridge - NBL)

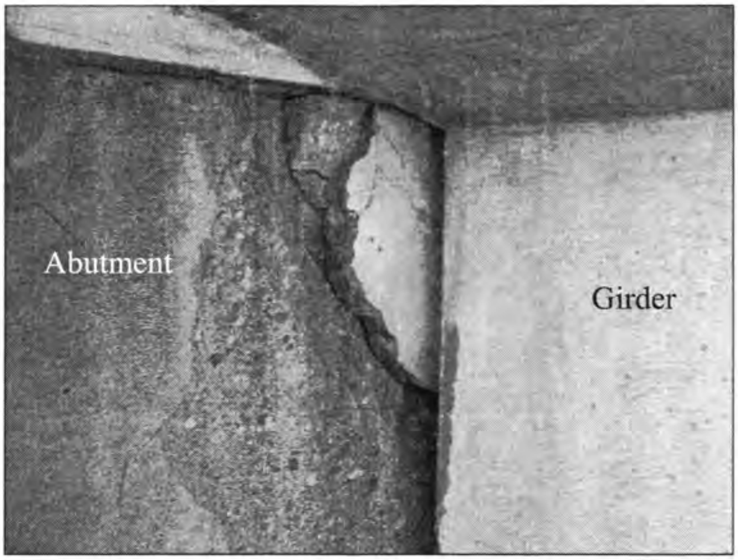


Figure 3.129 - Concrete Spalling at the Abutment (Polk Blvd. Bridge - SBL)

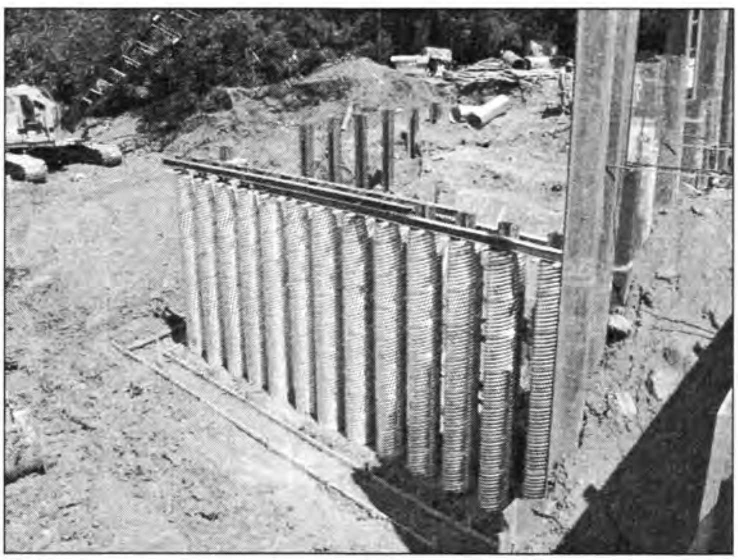


Figure 3.130 - Installation of H-Piles (Polk Blvd. Bridge - North End of NBL)

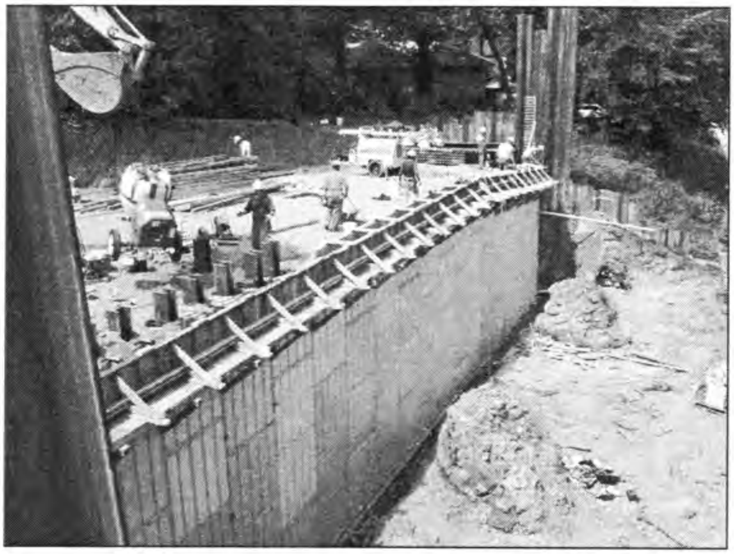


Figure 3.131 - Construction of the Retaining Wall (Polk Blvd. Bridge - South End of NBL)

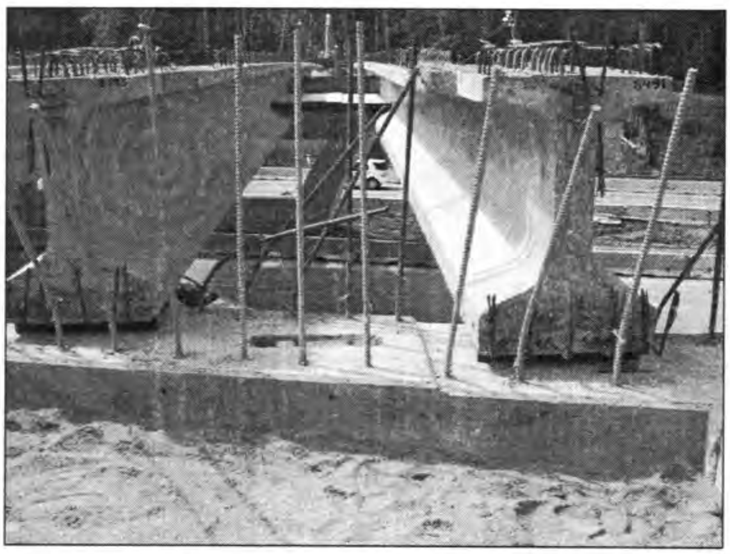


Figure 3.132 - Sand used as Granular Backfill Material (Polk Blvd. Bridge)

Figure 3.133 - No Porous Fill Surrounding the Subdrain (Polk Blvd. Bridge - North End NBL)

Figure 3.134 - H-Piles Being Driven Into the Ground (Polk Blvd. Bridge)

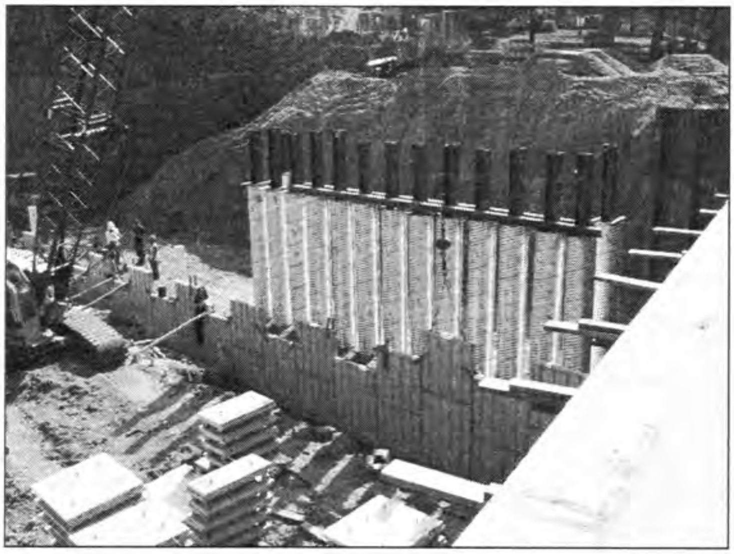


Figure 3.135 - Construction of the Retaining Wall (Polk Blvd. Bridge - South End of SBL)

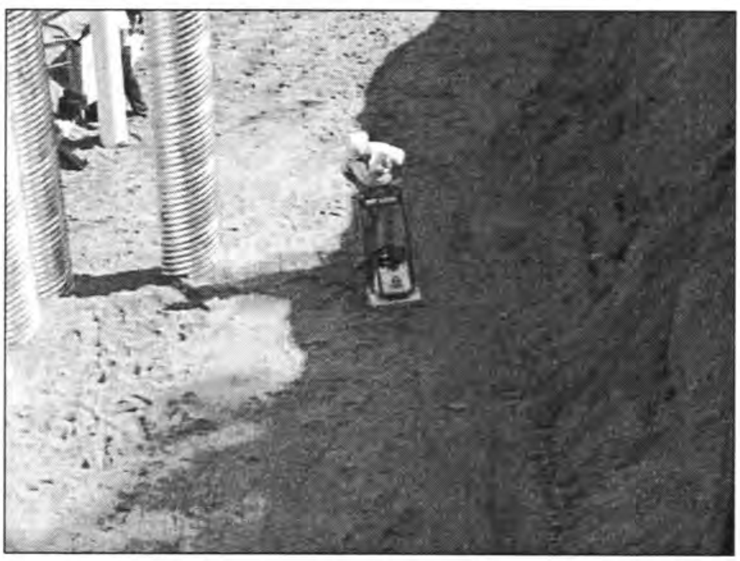


Figure 3.136 - Compaction of Retaining Wall Fill Material (Polk Blvd. Bridge - South End of SBL)

Figure 3.137 - DCP and Nuclear Gauge Tests (Polk Blvd. Bridge - North End of NBL)

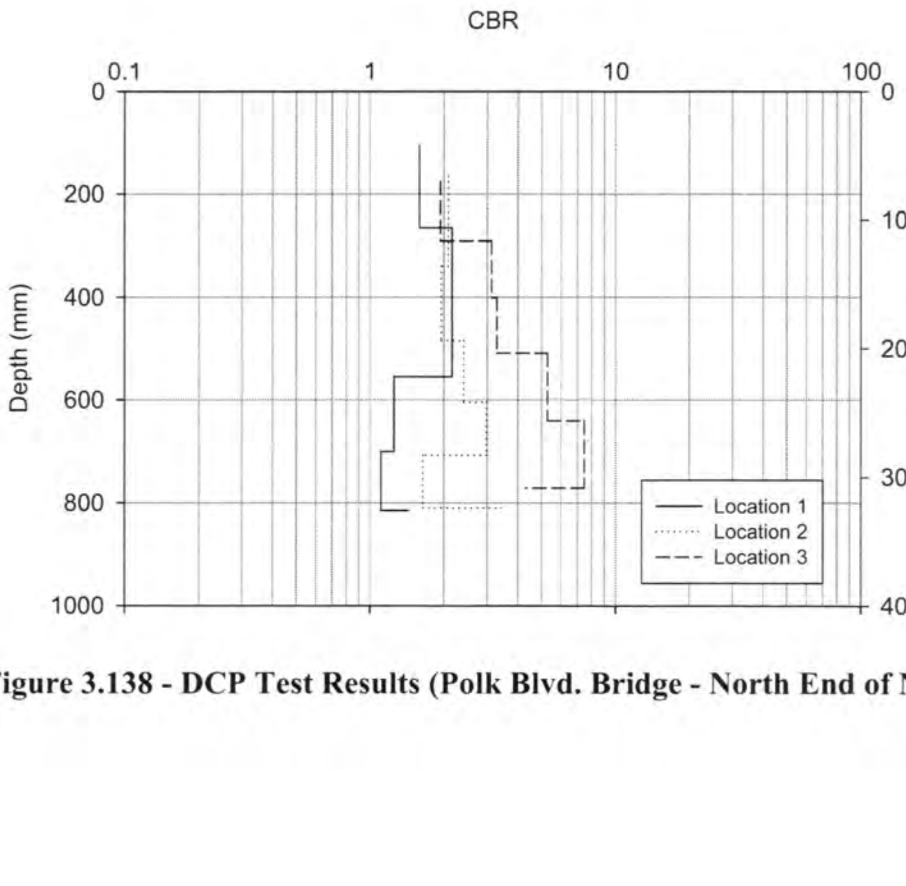


Figure 3.138 - DCP Test Results (Polk Blvd. Bridge - North End of NBL)

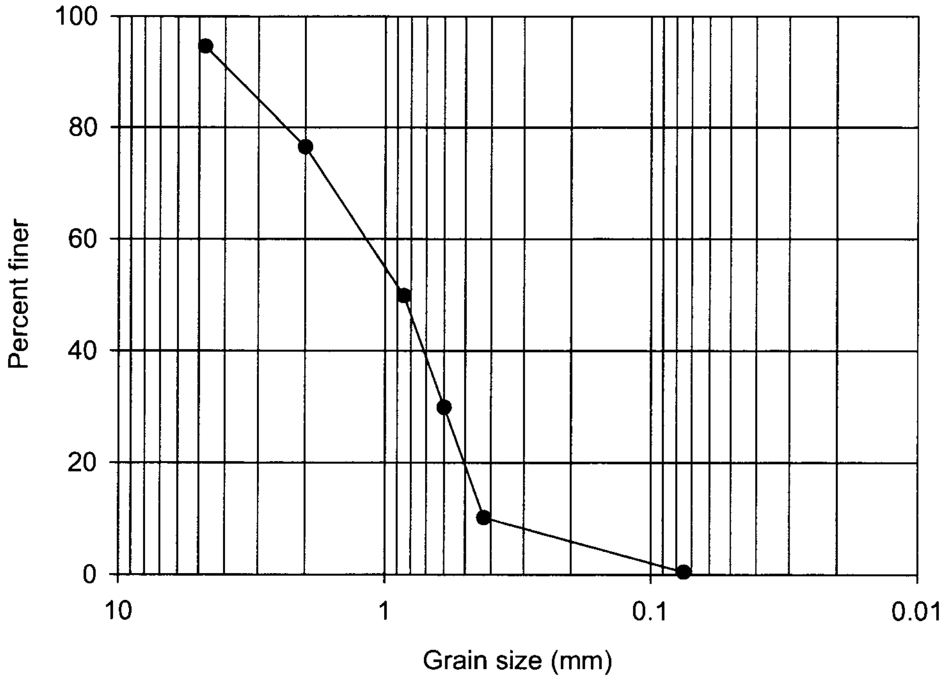


Figure 3.139 - Gradation of the Granular Bbackfill Material Classified as SP (Polk Blvd. Bridge)

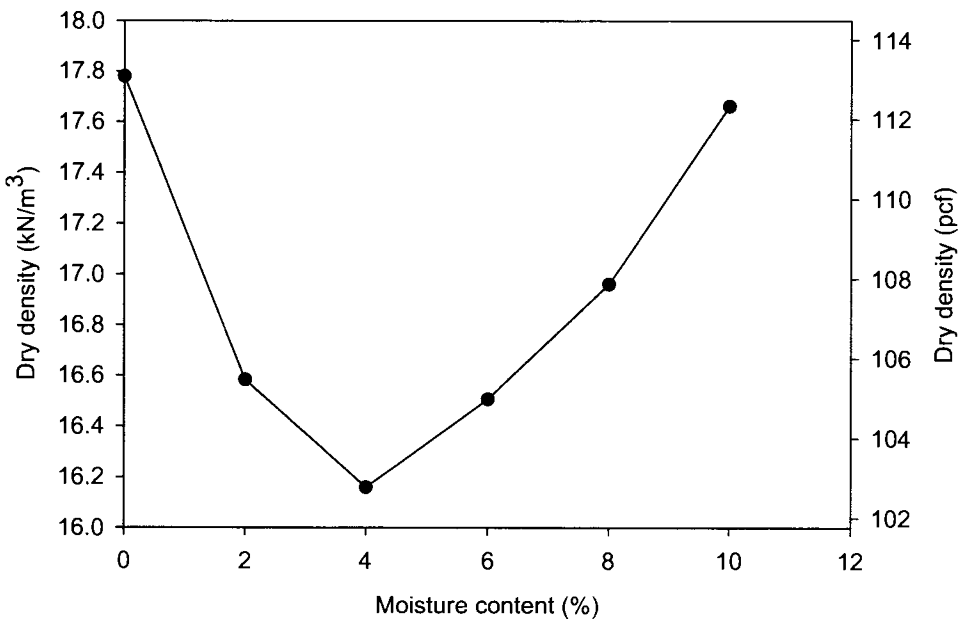


Figure 3.140. Dry Density – Moisture Relationship for Backfill Material (Polk Blvd. Bridge)

<u>19th Street Bridge over I-235 (District 1)</u>

The bridge was inspected in June 2003, and it was still open for traffic. This bridge has three spans and concrete slope protection at the bridge embankment. Differential settlement of 5.1 cm was observed at both ends of the bridge. Both ends of the bridge had an asphalt approach slab with visible longitudinal cracks (See Figure 3.141). When the bridge was visited again in October 2003, the girders and the backwall of the south bound lane were constructed (See Figure 3.142). Backfill materials samples were collected to perform laboratory tests.

Figure 3.143 shows the grain size distribution of the granular backfill material used behind the backwall. The backfill was classified as SP according to the USCS. The vibrating table test conducted showed that the minimum and maximum dry densities were 14.3 kN/m^3 and 18.2 kN/m^3 respectively, and the bulking moisture content ranged from 4% to 6% (See Figure 3.144). Compactibility of 0.86 was calculated indicating good compactibility.

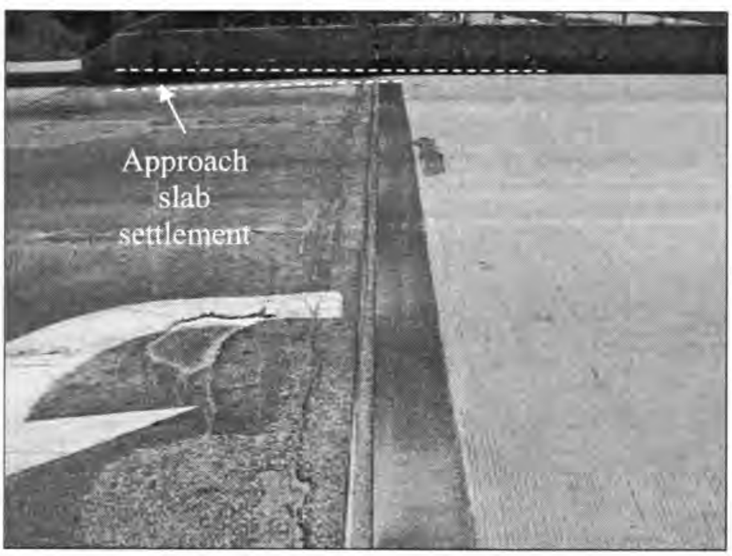


Figure 3.141 - Cracks at Asphalt Approach Slab with Visible Differential Settlement (19th Street Bridge)

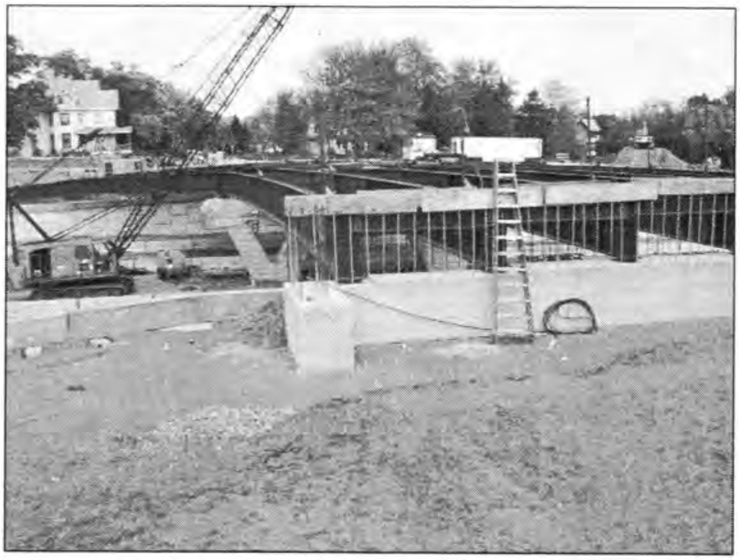


Figure 3.142 - Construction of Steel Girders and Backwall (19th Street Bridge)

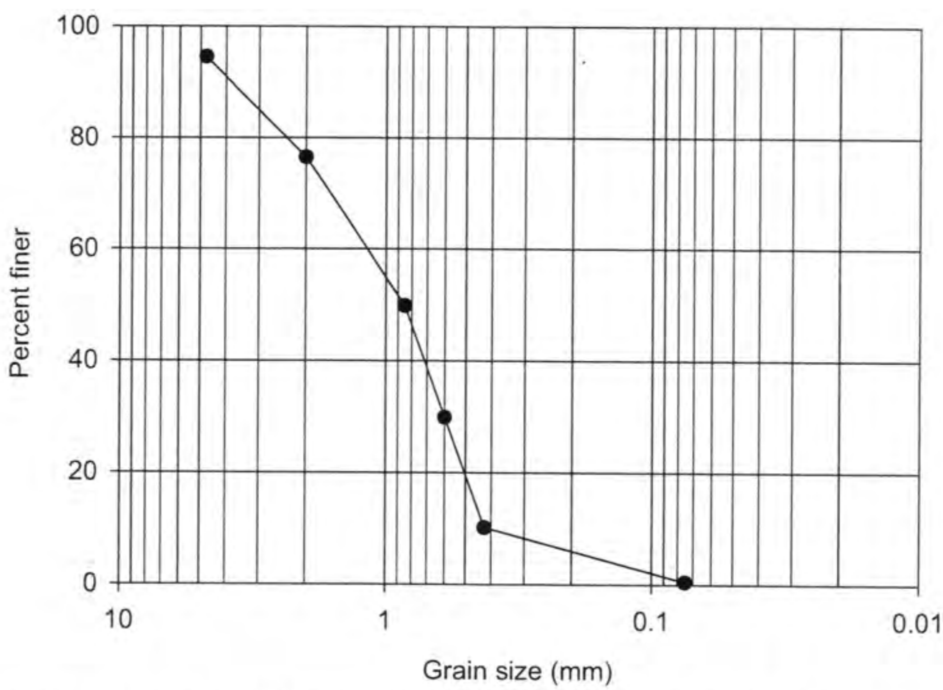


Figure 3. 143 - Gradation of Granular Backfill Classified as SP (19th Street Bridge)

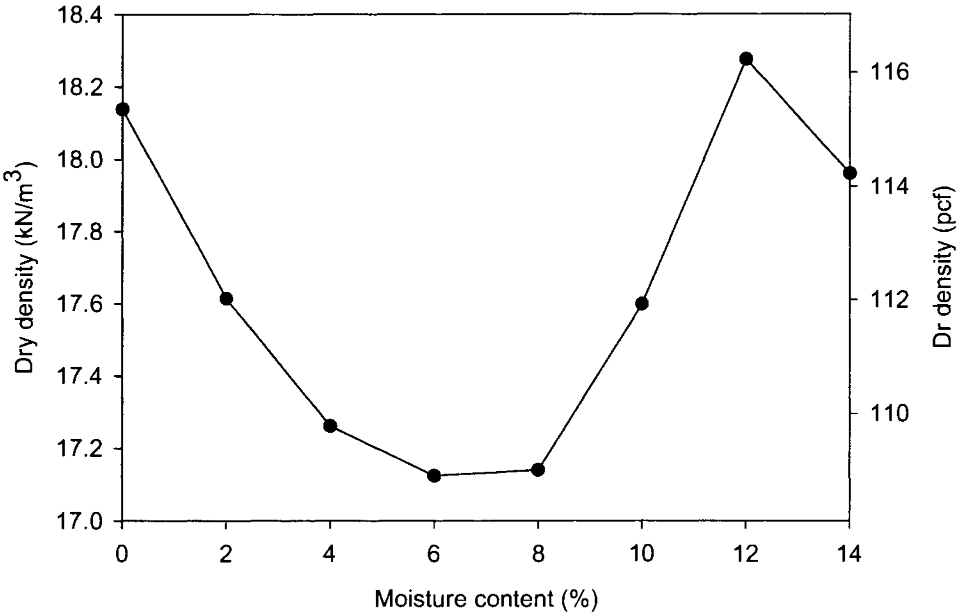


Figure 3.144 - Dry Density – Moisture Relationship for Granular Backfill (19th Street Bridge)

Pennsylvania Avenue Bridge over I-235 (District 1)

This bridge was inspected in June and October 2003. In June the center piers were still under construction as shown in Figure 3.145. In October, the concrete slope protection of the embankment under the bridge was constructed (See Figure 3.146). No porous backfill was placed around the subdrain. Figure 3.147 shows the outlet of the subdrain at the bottom of the bridge embankment surrounded by granular backfill. When inspected, the subdrain was filled with soil particles.

Figure 3.145 - Construction of Center Pier (Pennsylvania Avenue Bridge)

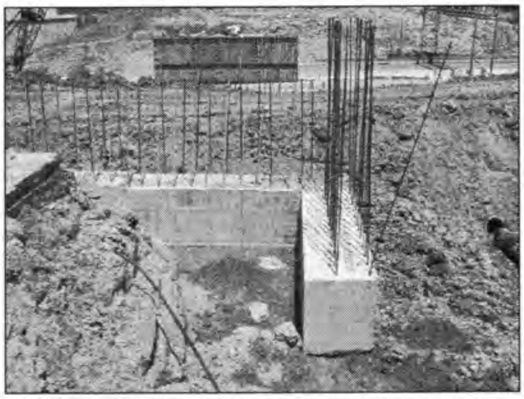
Figure 3.146 - Embankment Concrete Slope Protection (Pennsylvania Avenue Bridge)

Figure 3.147 - Subdrain Outlet at the Bottom of the Embankment (Pennsylvania Avenue Bridge)

East 12th Street Bridge over I-235 (District 1)

During the first inspection in June 2003, the center pier and the backwall of both ends were under construction (See Figure 3.148). As shown in Figure 3.148a, the soil near the south end of the north bound lane abutment appeared to be wet sandy soil. When inspected in October 2003, the construction of the bridge deck was in progress (See Figure 3.149). Figure 3.150 shows the reinforcements of the abutment which were 30.5 cm apart. Sample of backfill material was tested in the laboratory

Figure 3.151 shows the grain size distribution of the backfill material. The granular backfill obtained at the backwall of the south bound lane was classified as SP according to the USCS. Vibrating table test resulted in minimum and maximum dry densities of 15.1 kN/m^3 and 19.7 kN/m^3 and the bulking moisture content range of 3% to 5% moisture (See Figure 3.152). Compactibility of 1.227 was calculated indicating good compactibility of the backfill material.



(a) Backwall construction

(b) Center pier construction

Figure 3.148 - Construction of Backwall and Center pier (East 12th Street Bridge)

Figure 3.149 - Construction of Bridge Deck (East 12th Street Bridge - NBL)

Figure 3.150 - Abutment and Paving Notch Rebars Tied with Reinforcement from the Backwall (East 12th Street Bridge)

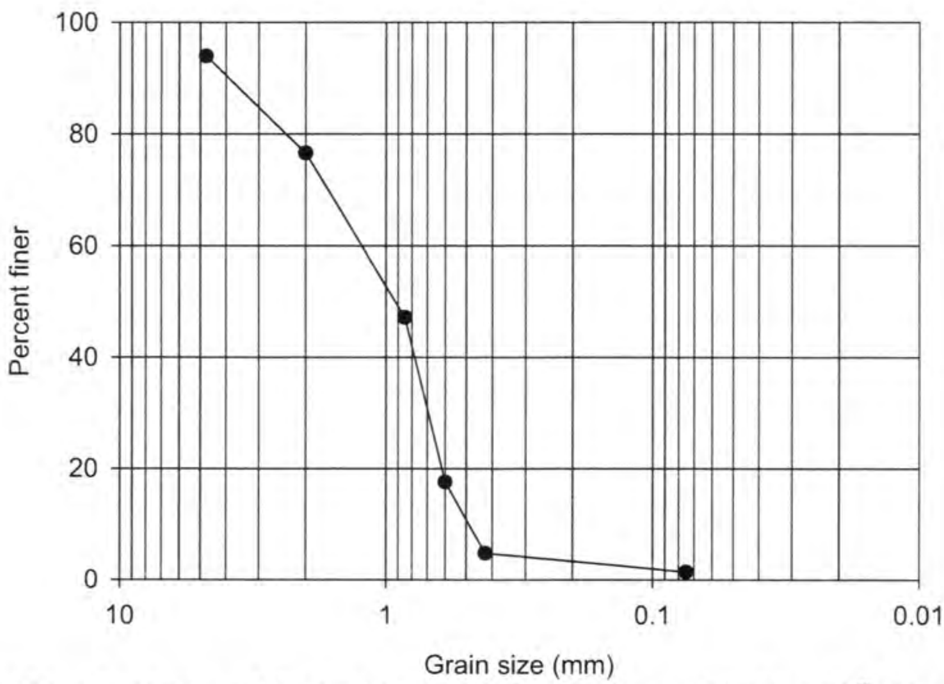


Figure 3.151 - Gradation of Granular Backfill Classified as SP (East 12th Street Bridge)

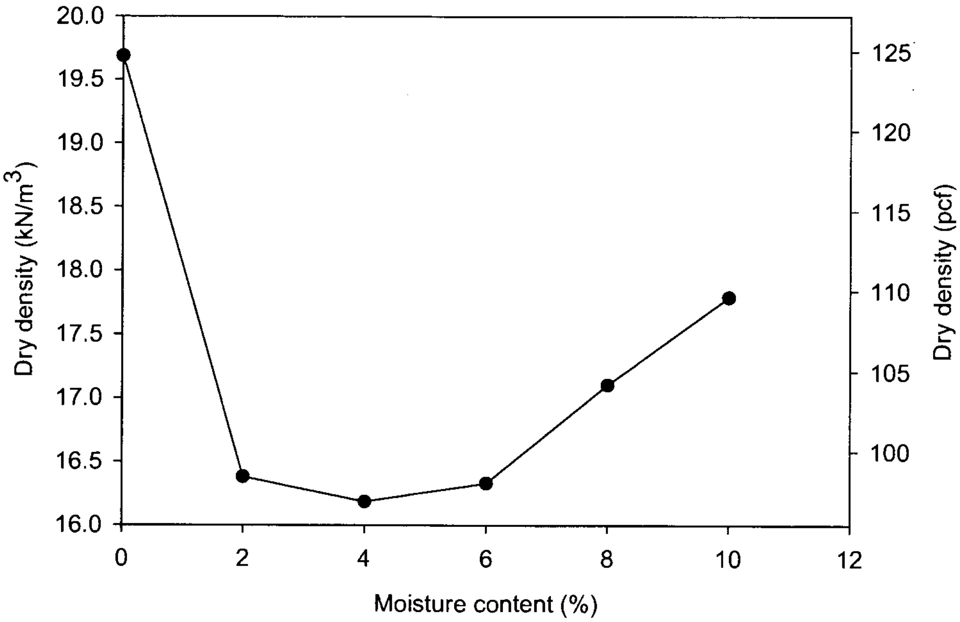


Figure 3.152 - Dry Density–Moisture Content Relationship for Granular Backfill (East 12th Street Bridge)

Euclid Avenue Bridge over I-235 (District 1)

During the field inspection in June 2003, the north bound lane bridge deck was under construction (See Figure 3.153) while the old south bound lane bridge was still open for traffic. The new bridge is a two span bridge with steel girders and integral abutments. Figure 3.154 shows the construction of the concrete slope protection of the embankment at the north end of the north bound lane. The slope protection was not yet constructed at the south end.

This bridge was also inspected twice in August 2003. During the first inspection, construction of the north bound lane was completed and construction of the south bound lane was taking place as shown in Figure 3.155. Figure 3.156 shows the end drain filled with soil particles. Construction of the backwall was taking place as shown in Figures 3.157 and 3.158, and the backwall reinforcements were at 30.5 cm spacing. During the second inspection in August 2003, construction of the abutment was still taking place. Poor construction of the approach slab at the north bound lane, which was recently completed and opened for traffic, was noticed. When the south bound lane was removed for replacement, the loose backfill behind the north bound lane abutment collapsed creating a void (See Figures 3.159 and 3.160).

The bridge was visited again in October 2003. The old south bound lane bridge was removed and the new bridge was under construction. The center pier construction was taking place, while the embankments and backwall were completed. Figure 3.161 also shows that porous backfill was used around the subdrain. The bridge embankments had a concrete slope protection as shown in Figure 3.162.

Figure 3.153 - Construction of Bridge Deck (Euclid Avenue Bridge)

Figure 3.154 - Construction of Slope Protection at the Embankment (Euclid Avenue Bridge)

Figure 3.155 - Construction of the South Bound Lane Bridge (Euclid Avenue Bridge)

Figure 3.156 - End Drain Outlet Filled with Soil (Euclid Avenue Bridge)

Figure 3.157 - Reinforcement for the Bridge Backwall (Euclid Avenue Bridge)

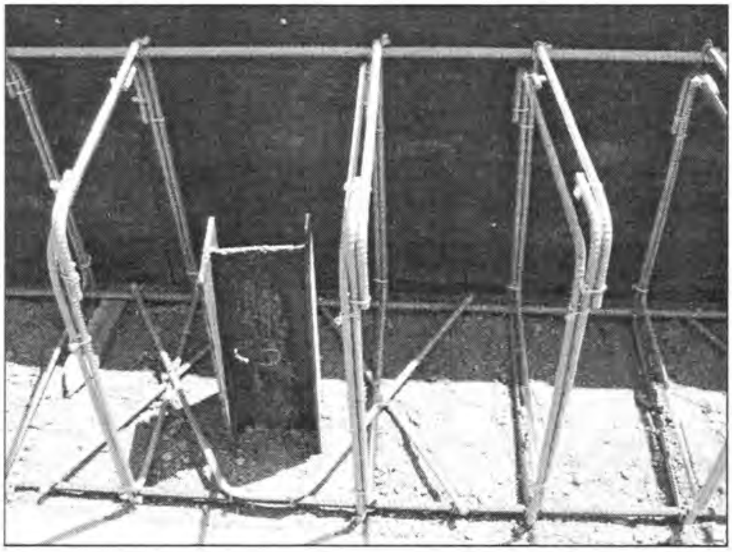


Figure 3.158 - H-Pile Embedded in the Bridge Backwall (Euclid Avenue Bridge)

Figure 3.159 - Abutment Reinforcement and Backfill Placed at the North Bound Lane Approach Slab (Euclid Avenue Bridge)

Figure 3.160 - A Closer Image of the Poorly Placed Granular Backfill (Euclid Avenue Bridge - NBL)

Figure 3.161 - Constructed Bridge Backwall with Visible Subdrain Surrounded by Porous Backfill (Euclid Avenue Bridge)

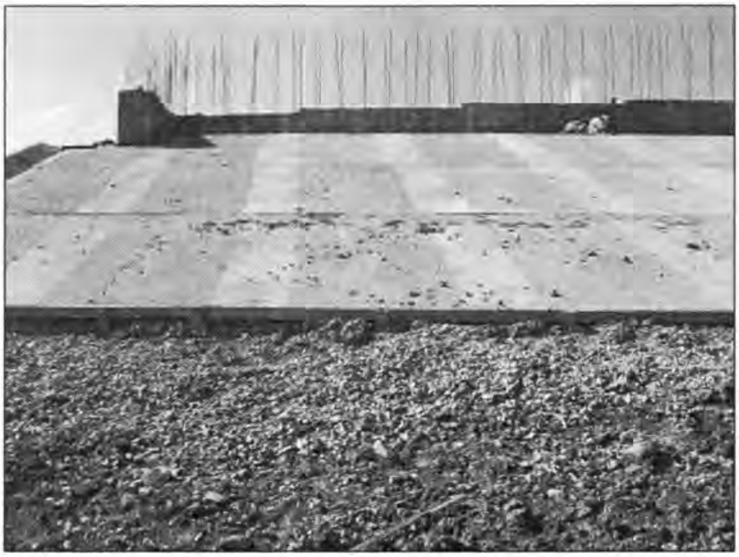


Figure 3.162 - Construction of Embankment Slope Protection (Euclid Avenue Bridge)

<u>Bridge over Union Pacific Railroad (District 3)</u>

This bridge has integral abutments and concrete girders. Gravel was used as slope protection at the embankment under the bridge. The foundation soil of this bridge was allowed to consolidate under the embankment weight for a year prior to construction of the bridge superstructure. Figure 3.163 shows the granular backfill poorly placed behind the abutment. As shown in Figure 3.164, no porous backfill was used around the subdrain. Figure 3.165 illustrates the poor construction of the paving notch with honey combing.

The backfill material sample obtained at the bridge site, which had a moisture content of 4.1%, was classified as SP according to the USCS. Figure 3.166 shows the grain size distribution of the granular backfill material used behind the abutment. Coefficient of permeability of 0.01 cm/s was measured using Constant Head Permeability Test (ASTM D 2434). When compared to the values reported by Das (1998), the measured coefficient of permeability was within the range of coarse and fine sand.

Figure 3.163 - Poorly Placed Granular Backfill (Bridge over Union Pacific Railroad)

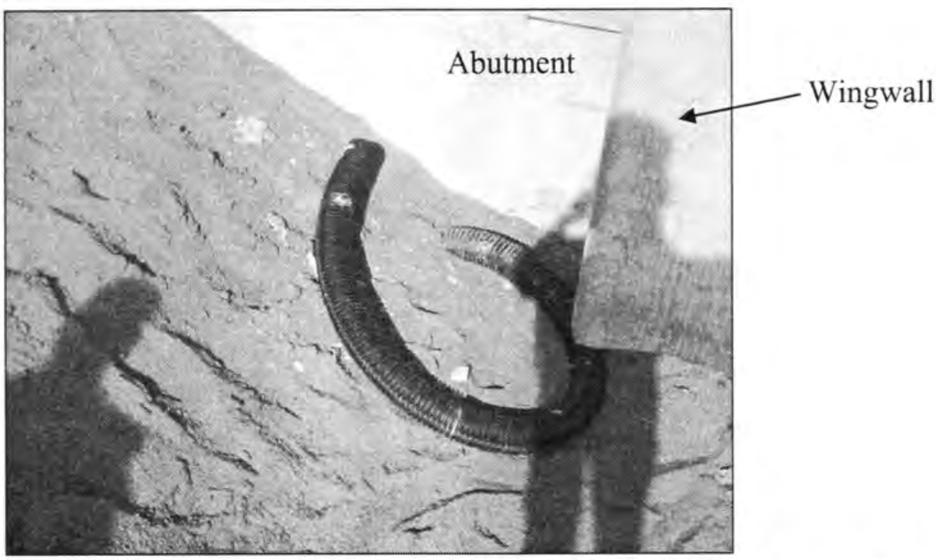


Figure 3.164 - Granular Backfill Surrounding the Subdrain (Bridge over Union Pacific Railroad)

Figure 3.165 - Poor Construction of Paving Notch (Bridge over Union Pacific Railroad)

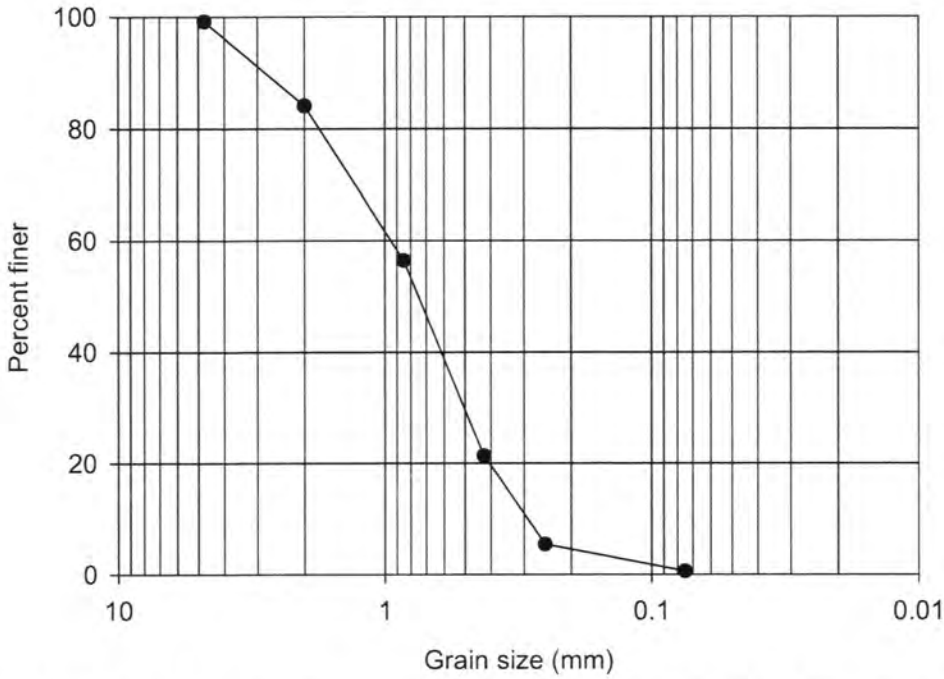


Figure 3.166 - Gradation of Granular Backfill Classified as SP (Bridge over Union Pacific Railroad)

Bridge No. 57.6R030 (District 6)

The bridge was under construction when inspected in August 2003. The paving notch was declined allowing for the approach slab to slide off the paving notch (See Figure 3.167). Figure 3.168 shows the grain size distribution of the granular backfill material used at this bridge, which was classified as SP according to the USCS.

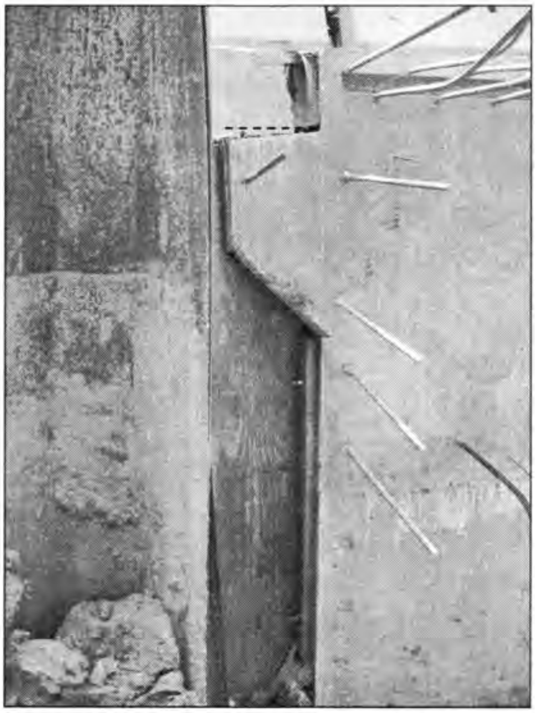


Figure 3.167 - Sloping Paving Notch (Bridge No. 57.6R030)

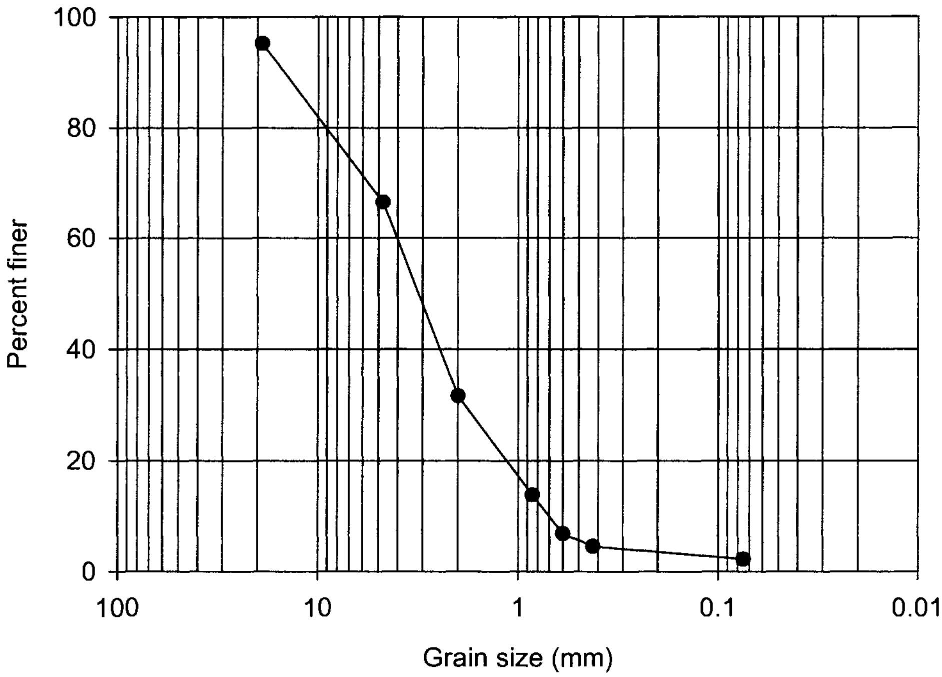


Figure 3.168 - Gradation of Granular Backfill Classified as SP (Bridge No. 57.6R030)

3.2.3 Maintenance and Rehabilitation Practices

Bridge at U.S. 218 Crossing Rail Road (District 5)

This bridge has three spans with steel girders and non-integral abutments. The bridge was grouted due to void development under the approach slabs (See Figure 3.169). During the field inspection, replacement of both approach slabs and paving notch of the north bound lane was in progress; however, the south bound lane was still open for traffic. These approach slabs were replaced due to severe erosion and void development under the approach slabs which led to excessive settlement and cracking at the bridge approaches. At the south end, the backfill was already placed and compacted in 200 mm lifts using a vibratory base plate. Granular backfill similar to the backfill observed at other bridges was used. The width of the newly constructed paving notch was 30.5 cm according to the old Iowa DOT specifications with an expansion joint width of 6.4 cm (See Figure 3.170). The old bridge did not have a subdrain, and even though the backfill material was replaced, no new subdrain was added. Silty clay was used at the embankment with no slope protection. At the north end, construction of the abutment was still in progress (See Figure 3.171). The backfill material was not placed and no subdrain will be added at this end.

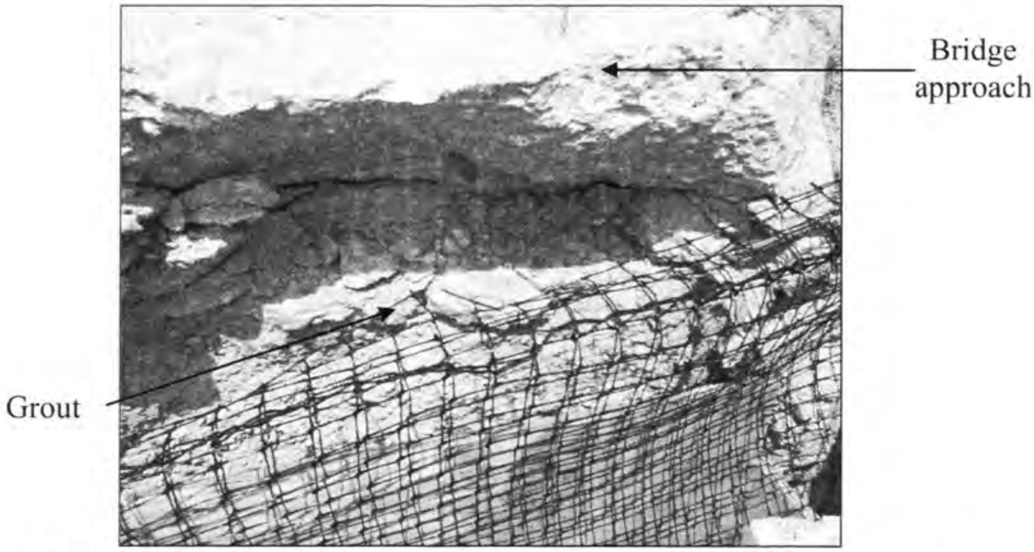


Figure 3.169 - Grout Pumped under Existing Approach Slab (Bridge at U.S. 218 Crossing Rail Road - South End of SBL)

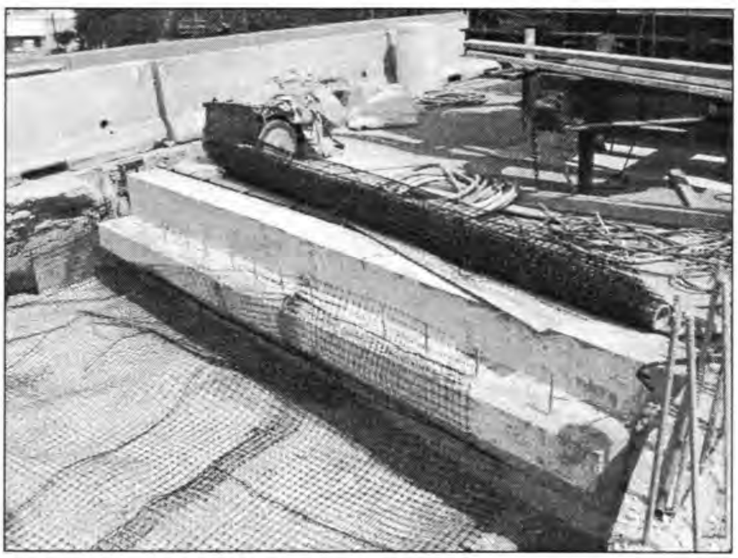


Figure 3.170 - Construction of New Bridge Approach (Bridge at U.S. 218 Crossing Rail Road - South End of NBL)

Figure 3.171 - Abutment Reinforcement (Bridge at U.S. 218 Crossing Rail Road - North End of NBL)

URETEK Inc. Maintenance Method

<u>Bridge No. 76.8065 (U.S. 65 over Des Moines River)</u>

The bridge is a seven span bridge with concrete girders and non-integral abutments. At the north end of the north bound lane, silty sand was used to build the bridge embankment with no slope protection under the bridge. As a result erosion of the bridge embankment was observed. At the abutment side, rocks were placed to prevent further erosion as shown in Figure 3.172.

At the south end of the north bound lane, differential settlement of 3.8 cm between the approach slab and the bridge deck and 6.4 cm between the approach slab and the wingwall was measured The expansion joint was in a satisfactory condition. At the bridge embankment rocks were placed to prevent further erosion (See Figure 3.173). Erosion was observed at the abutment sides.

At the south end of the south bound lane, the strip seal was cut short causing water to flow around the bride eroding the abutment sides. Settlement and erosion of the bridge embankment were also noticed. At the north end of the south bound lane, 8.9 cm differential settlement was observed (See Figure 3.174). No slope protection was used and erosion of the embankment was noted.

The bridges approach at the north end of the south bound lane was cored and a 3.2 cm void at the edge line and 5.1 cm void at the center line was detected. Since only the north end will be maintained, the south end was not cored.

The approach slabs profiles obtained for the both bound lanes are shown in Figures 3.175 and 3.176. The settlement of the approach slab at the north end of the south bound lane was 3 cm at 1.5 m away from the bridge, while the south end approach slab settled 8.9 cm at 15 m away from the bridge. The differential settlement between the approach slab and the roadway of both ends of the north bound lane was approximately 1.3 cm. The gradients for the north and south end of the north bound lane approach slabs are 0.012 and -0.019 respectively, while the gradients for the north and south end of the south bound lane approach slabs are 0.01 and -0.019 respectively.

The snake camera was used to inspect the subdrain at the north end of the north bound lane. At 1.8 m inside the subdrain, the subdrain was partially collapsed and the snake camera could not be pushed through. The brushes mounted on the camera had to be removed in order to go further into the subdrain. At 3.1 m inside the subdrian, the subdrain was completely collapsed, and the snake camera could not be pushed further. The first 3 m observed were dry and with no water or fine particles indicating that the drain is not functioning.

Figure 3. 172 - Rocks Placed at the Abutment Side to Prevent Erosion (Bridge No. 76.8065 - North End of NBL)

Figure 3.173 - Rocks Placed at the Bridge Embankment to Prevent Further Erosion (Bridge No. 76.8065 - South End of NBL)

Figure 3.174 - Settlement of Approach Slab (Bridge No. 76.8065 - North End NBL)

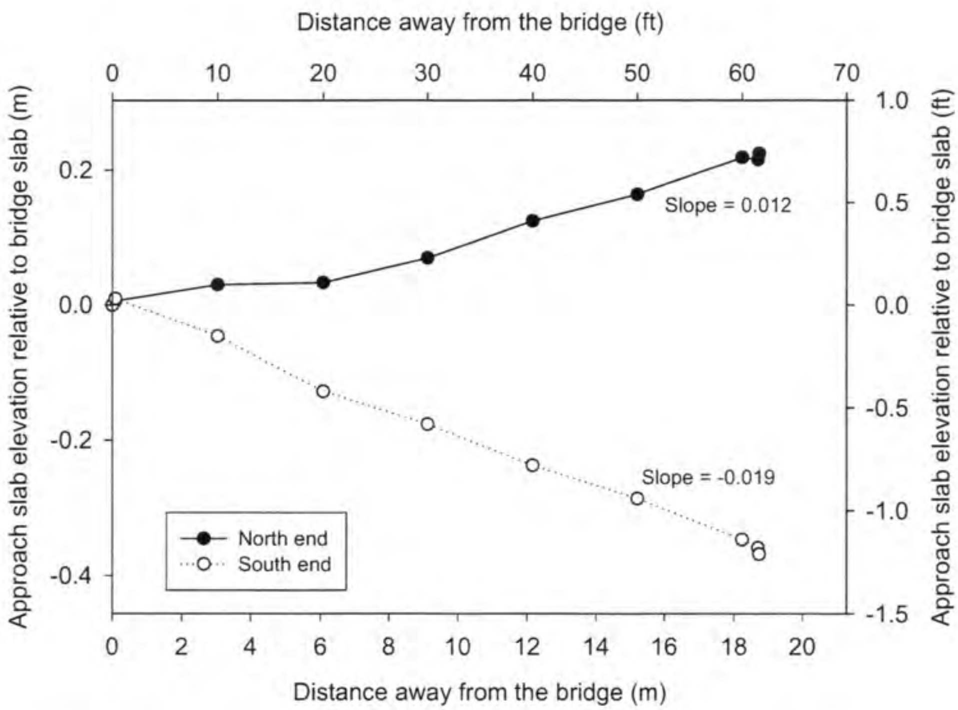


Figure 3.175 - Profiles of the Approach Slab Relative to the Bridge Deck (Bridge No. 76.8065 - NBL)

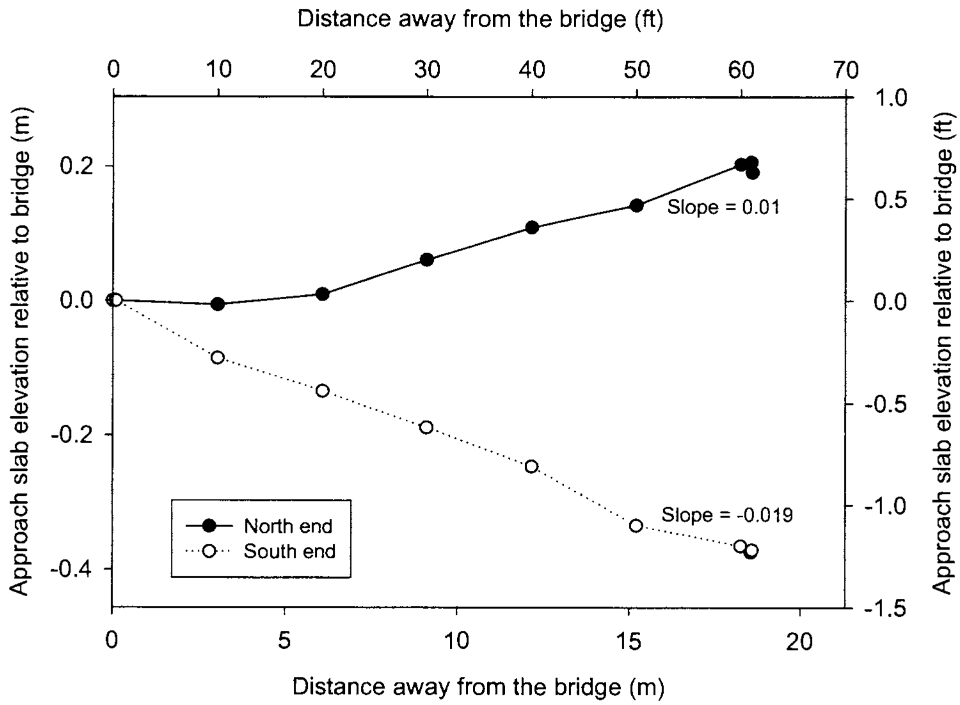


Figure 3.176 - Elevation of the Bridge Approach Relative to the Bridge Deck (Bridge No. 76.8065 - SBL)

URETEK Inc. performed maintenance demonstration on the right lane of the north end of the north bound lane. As shown in Figure 3.177 three columns of 1.6 cm (5/8 in.) holes were drilled between the edge line and the centerline of the road. The columns were measured to be 1.3 m apart. A fourth column of holes was drilled at the shoulder. The holes on each column were spaced at 1.5 m. All the holes were drilled on the approach slab and extended 15.2 m away from the bridge (See Figures 3.178 and 3.179). A dial gauge was placed after the 'CF' joint, 18.2 m away from the abutment, to measure any change in the elevation of the approach relative to the road (See Figure 3.180). After drilling was completed, pumping of high density polyurethane was performed. Some holes were redrilled after the material has hardened to inject more material as needed (See Figures 3.181and 3.182 While injecting the material, readings were taken to measure the change in elevation along the slab and observe how much the approach slab is raised.

The profile of the approach slab at 3 m intervals was measured before and after injecting the polyurethane material. The demonstration was not completed due to a thunder storm; however, the approach slab was raised approximately 0.6 cm from 12.2 m to 18.2 m from the abutment, which is where the material was injected. Due to weather condition, the URETEK crew did not inject the material and raise the approach slab from 0 to 9.1 m from the abutment (See Figure 3.183).

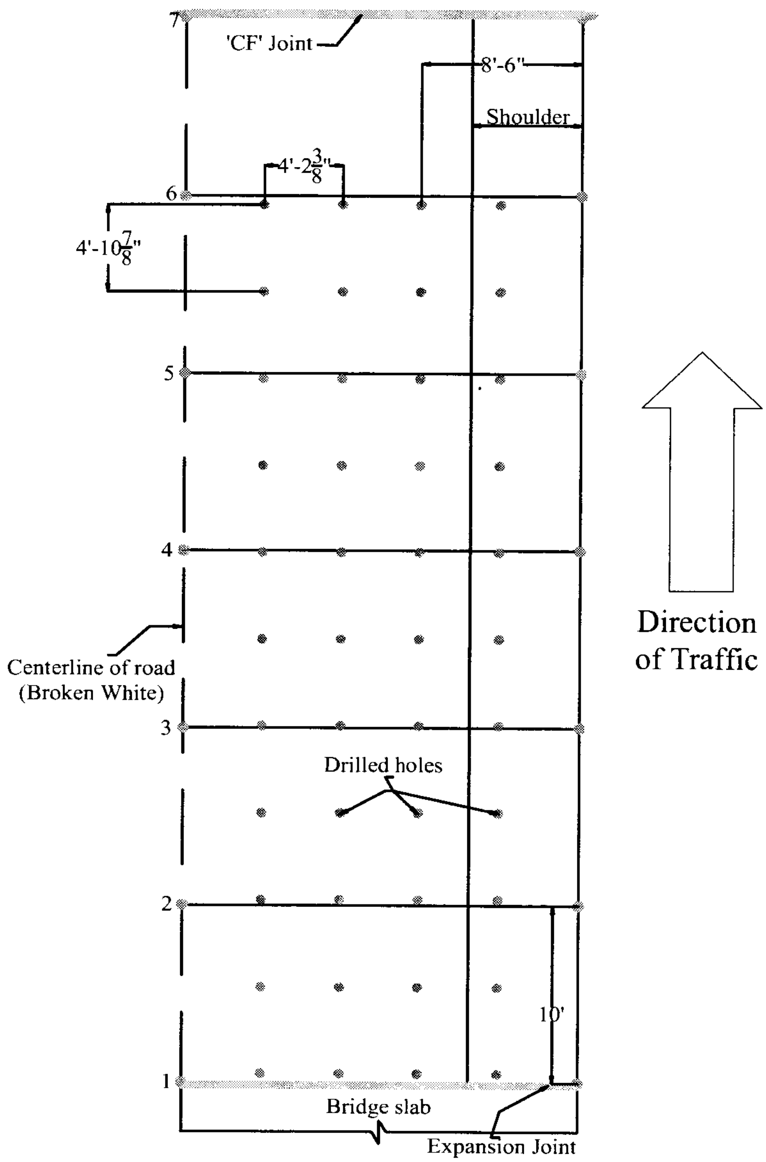


Figure 3.177 - Location of Holes Drilled on the Bridge Approach Slab (Bridge No. 76.8065)

Figure 3.178 - Holes Drilled at the Approach Slab (Bridge No. 76.8065)

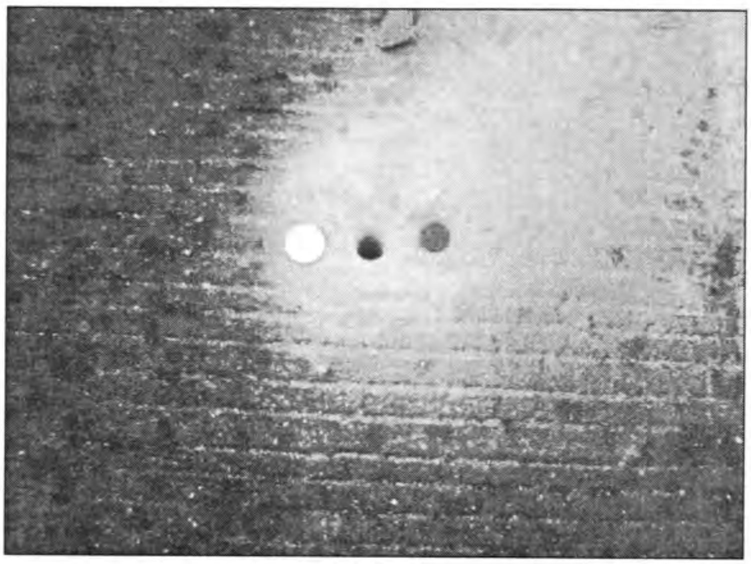


Figure 3.179 - Holes Drilled for Injecting Expansive Material under the Approach Slab (Bridge no. 76.8065)

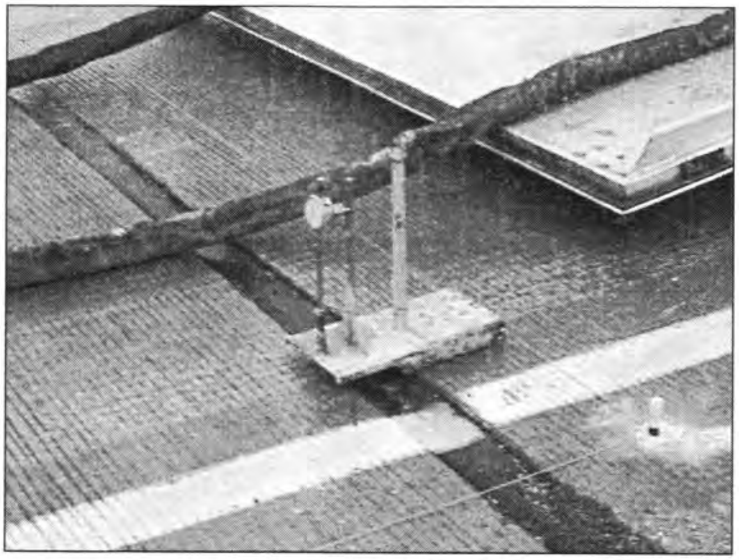


Figure 3.180 - Dial Gauge to Measure Change in Approach Slab Elevation (Bridge No. 76.8065)

Figure 3.181 - Injecting High Density Polyurethane under the Approach Slab (Bridge No. 76.8065)

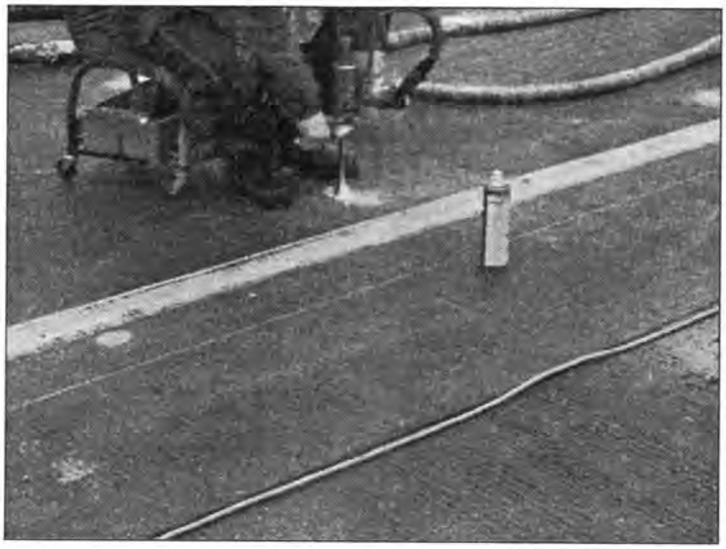


Figure 3.182 - Steady Injection until Material Leaks Out of Hole (Bridge no. 76.8065)

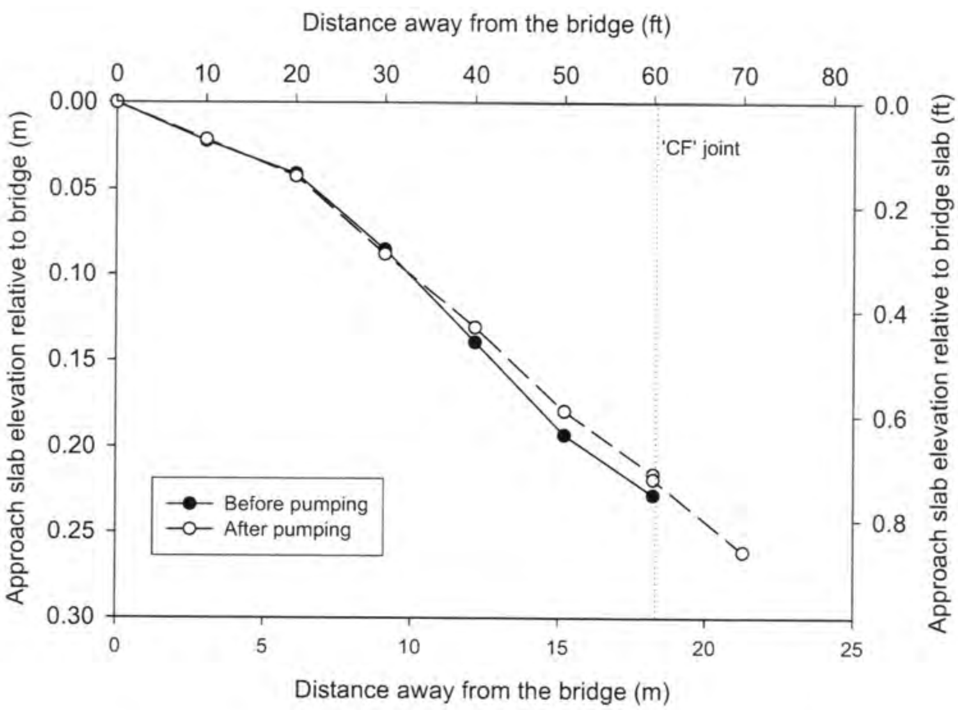


Figure 3.183 - Elevation of Approach Slab Relative to the Bridge before and after Injecting the Polyurethane (Bridge No. 76.8065)

Bridge No. 1783.6018

This bridge, which is a three span bridge and constructed in 1997, has integral abutments, concrete girders, and crosses over a railroad. According to the maintenance report, the approach slab was replaced in July 1999.

At the west end of the east bound lane, differential settlement between the bridge approach and the bridge deck at the wingwall was approximately 3.8 cm (See Figure 3.184). Furthermore, the expansion joint sealer was deteriorated exposing the flexible foam filler material which was poorly sealing the expansion joint leaving a gap for water to flow around the bridge as shown in Figure 3.185. The bridge embankment has a slope protection which was in good condition; however, the embankment settled 8.9 cm as shown in Figure 3.186. A 2.5 cm gap between the bridge embankment and the bridge abutment was measured (See Figure 3.187).

At the east end of the east bound, differential settlement at the wingwall was 2.5 cm (See Figure 3.188). Similar to the west end, the sealer material at the expansion joint was deteriorated exposing the flexible foam material, which poorly sealed the expansion joint. In addition, concrete spalling near the expansion joint was observed (See Figure 3.189).

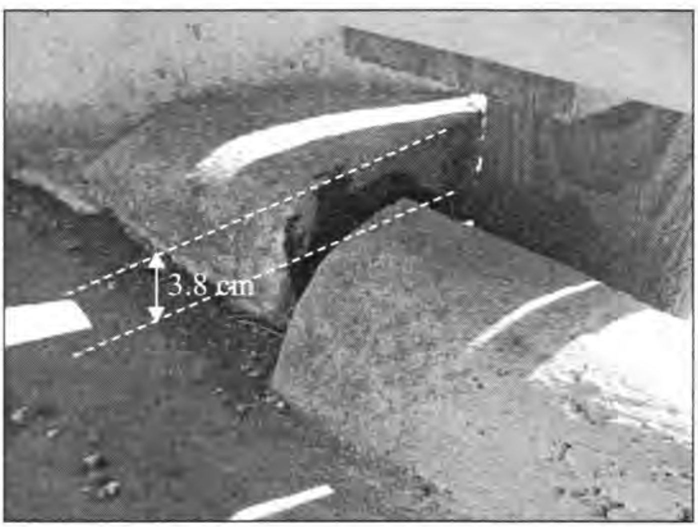


Figure 3.184 - Differential Settlement between the Bridge Approach and the Approach Slab (Bridge No. 1783.6018 - West End of EBL)

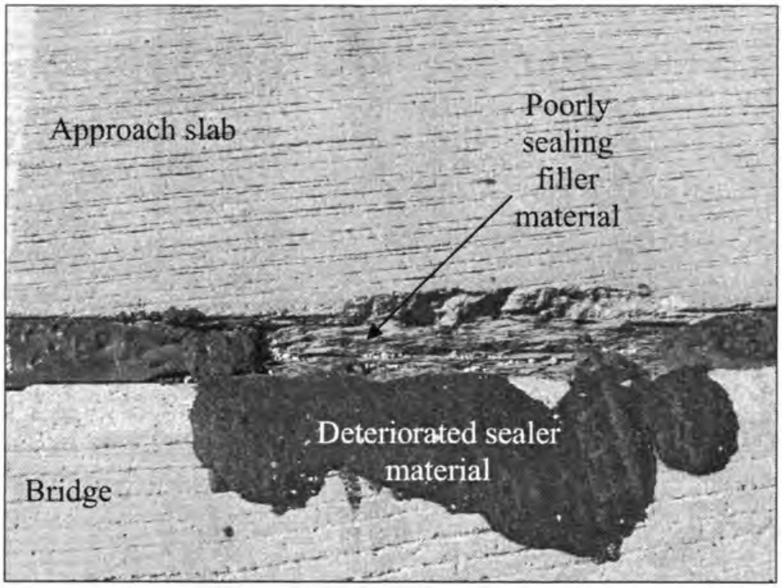


Figure 3.185 - Deterioration of the Expansion Joint Sealer Exposing the Poorly Sealing Filler Material (Bridge No. 1783.6018 - West End of EBL)

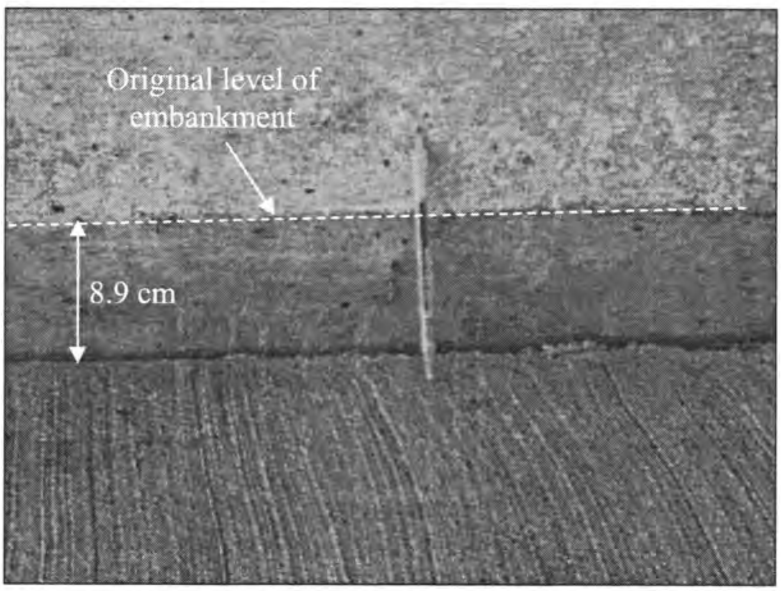


Figure 3.186 - Settlement of the Bridge Embankment (Bridge No. 1783.6018 - West End of EBL)

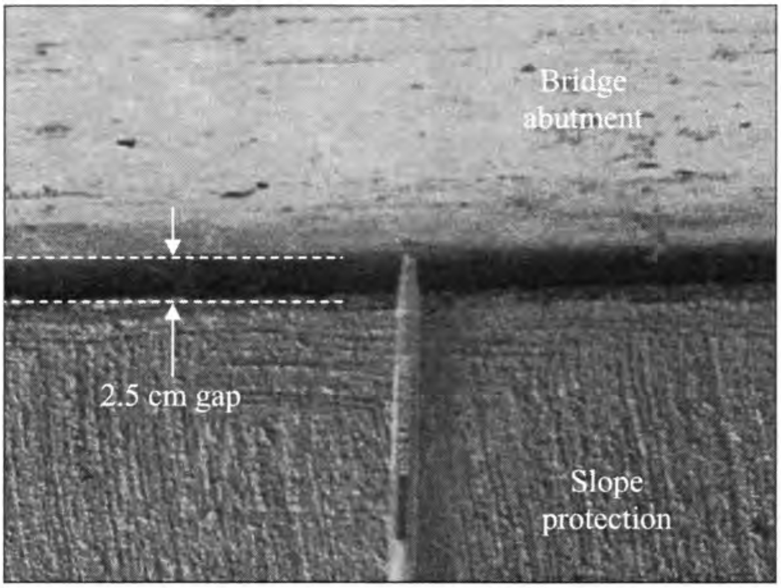


Figure 3.187 - Top View Showing a Gap between the Bridge Embankment and the Bridge Abutment (Bridge No. 1783.6018 - West End of EBL)

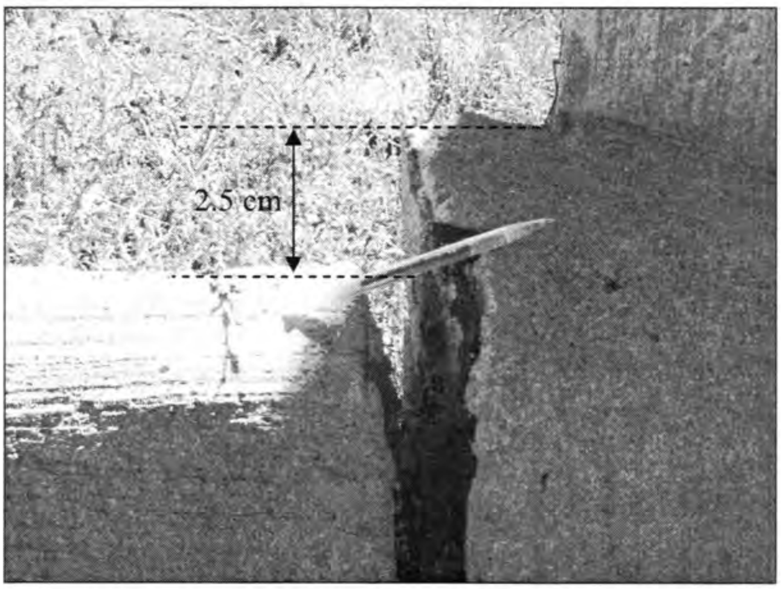


Figure 3.188 - Differential Settlement between the Bridge Approach and the Approach Slab (Bridge No. 1783.6018 - East End of EBL)

Figure 3.189 - Poorly Sealed Expansion Joint with Concrete Spalling (Bridge No. 1783.6018 - East End of EBL)

URETEK Inc. injected polyurethane material under both approach slabs at the east bound lane to alleviate the bump caused by excessive settlement. The profiles of the approach slabs before and after injecting the expansive polyurethane were measured and presented in Figure 3.190. Both approaches were lifted approximately 4.5 cm at 6 m away from the bridge; however, both approach slabs were lifted approximately 1.3 cm higher than the bridge deck level creating a bump.

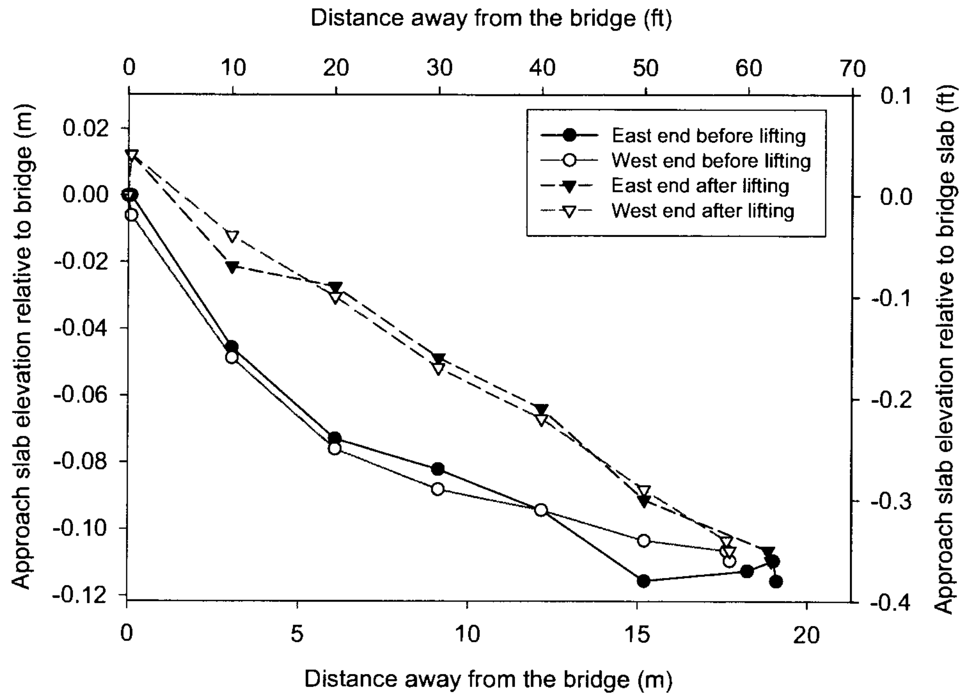


Figure 3.190 - Approach Slabs Elevations before and after Injecting the Expansive Polyurethane (Bridge No. 1783.6018 - EBL)

U.S. 65 Maintenance Project

<u>Bridge No. 7773.0R065 (U.S. 65 over IA 5)</u>

This is a two span bridge with steel girders and non-integral abutments. At the north end of the south bound lane, settlement of the approach slab was observed. Furthermore, a 21.6 cm void under the approach slab was observed as shown in Figure 3.191. The strip seal of the expansion joint was cut short and filled with soil. The end drain was in a satisfactory condition

At the south end of the south bound lane, differential settlement of 2.5 cm was measured at the wingwall. A 5.1 cm void was observed under the approach slab (See Figure 3.192). The bridge embankment, which had a gravel overlay for erosion control, experienced no settlement. Gravel was also used along the abutment sides to control erosion (See Figure 3.193). The end drain was in a satisfactory condition as shown in Figure 3.194.

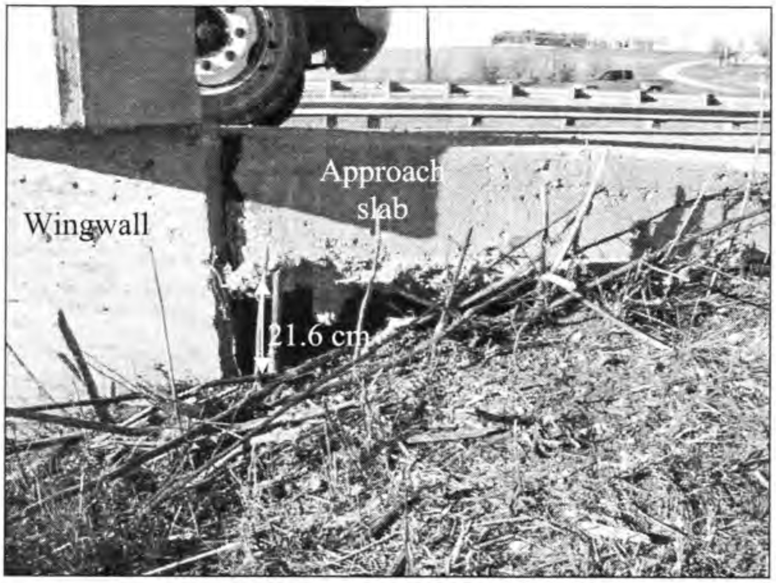


Figure 3.191 - Void under the Approach Slab (Bridge No. 7773.0R065 - North End of SBL)

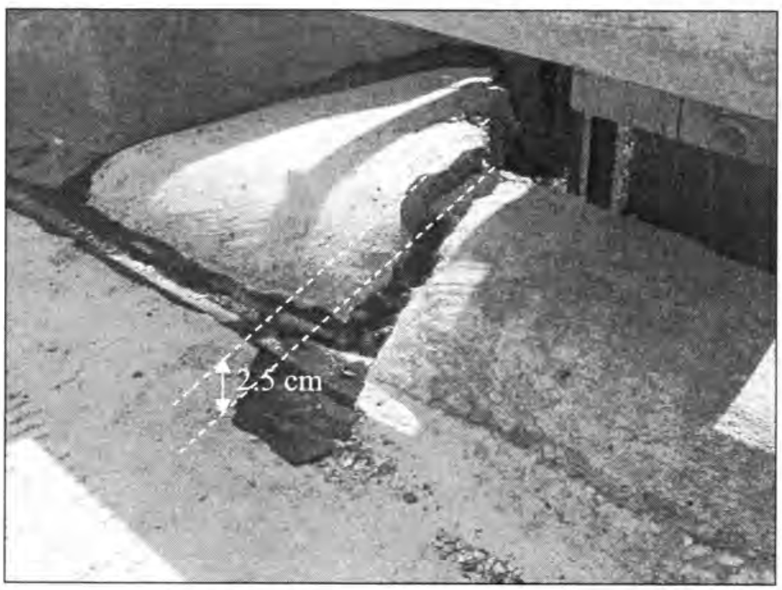


Figure 3.192 - Differential Settlement between the Bridge Deck and the Bridge Approach (Bridge No. 7773.0R065 - South End of SBL)

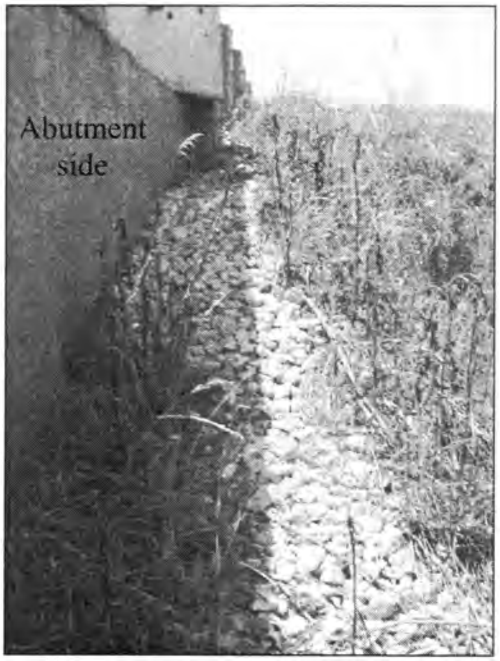


Figure 3.193 - Gravel Placed at the Sides of the Abutment to Control Erosion (Bridge No. 7773.0R065 - South End of SBL)

Figure 3.194 - End Drain in a Satisfactory Condition (Bridge No. 7773.0R065 - South End of SBL)

Before starting the maintenance at U.S. 65, Iowa DOT cored the approach slabs of bad performing bridges to measure the size of the developed void. These approach slabs were cored twice, one at the edge line and one at the center line, of the right lane at about 38 cm from the bridge. As the cored concrete dropped into the void, the distance relative to the original slab level was measured to estimate the void size. The concrete sample was then removed and the core was filled with expansive foam.

At this bridge, the approach slab of the north end of the north bound lane was cored and a 5.1 cm void was measured. However, due to the large void observed at the edge of the approach slab (See Figure 3.191), a third core was taken at the center line 5.5 m away from the bridge but no void was detected. The thickness of the cored concrete was about 26.7 cm

The elevation of the approach slab relative to the bridge deck at the edge line of the right lane was measured. A straight line connecting the elevation of the bridge deck to that of the roadway was assumed to be the original profile of the approach slab. Bridge approach differential settlement was estimated using the difference between the measured profile and the original profile.

At this bridge, profiles of the north and south ends of the south bound lane were obtained (See Figure 3.195). Differential settlement of 1.3 cm was estimated between the approach slab and the roadway at both ends of the south bound lane. Both profiles slope away from the bridge indicating settlement of either the embankment material or the foundation soil.

The subdrains of the north bound lane were inspected using the snake camera to examine the bridge subdrain (See Figures 3.196 and 3.197). At the north end, water and mud were observed within the first 2.7 m inside the subdrain, which indicate that the subdrain was still functioning (See Figures 3.198 and 3.199). The snake camera was pulled out, cleaned, and reinserted, however, the camera could not be pushed beyond 4.4 m where the pipe collapsed. At the south end the subdrain was completely dry, and no water or fines were observed. The camera could not be pushed beyond 3 m into the subdrain because of subdrain collapse.

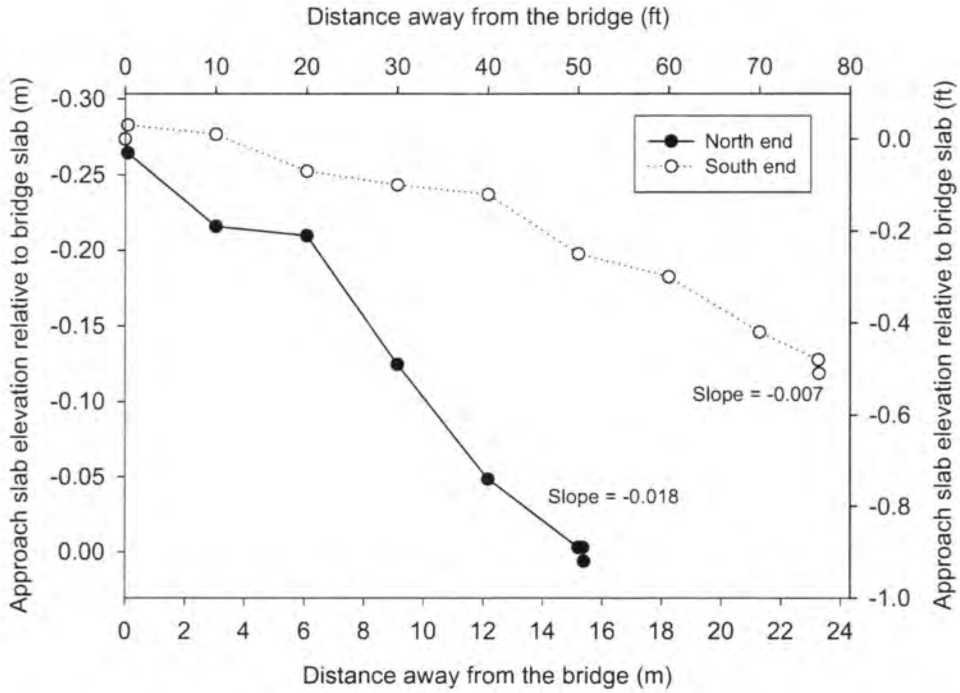


Figure 3.195 - Profiles of the Approach Slab Relative to Bridge Deck (Bridge No. 7773.0R065 - SBL)

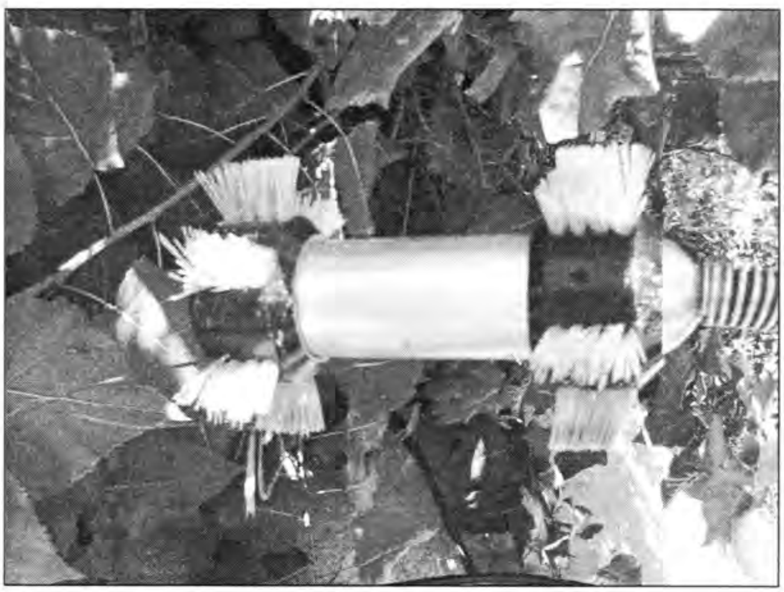


Figure 3.196 - Snake Camera used to Inspect Subdrains (Bridge No. 7773.0R065)

Figure 3.197 - Snake Camera Control Unit (Bridge No. 7773.0R065)

Figure 3.198 - Subdrain Prior to Insertion of Snake Camera (Bridge No. 7773.0R065 - North End of NBL)

Figure 3.199 - Snake Camera Covered with Mud (Bridge No. 7773.0R065 - North End of NBL)

Bridge No. 7774.0L065 (U.S. 65 over Avon Road)

The bridge is a three span bridge with steel girders and non-integral abutments. At the north end of the south bound lane, bridge approach differential settlement was 2.5 cm. No settlement of the bridge embankment, which was covered by aggregate, was observed (See Figure 3.200). At the south end, differential settlement at the bridge approach was observed. In addition, water ponding at the bottom of the bridge embankment was noted. The inlet of the surface drain was in a satisfactory condition.

Approach slab coring at the south end of the south bound lane indicated a 1.3 cm void at both the center line and the edge line of the right lane.

The profile of the south end approach slab was obtained which showed a wavy approach slab. The profile shows a maximum settlement of 4.8 cm at 6.1 m away from the bridge. The original profile of the bridge is horizontal (See Figure 3.201).

Figure 3.200 - Aggregate Slope Protection at the Bridge Embankment (Bridge No. 7774.0L065 - North End of NBL)

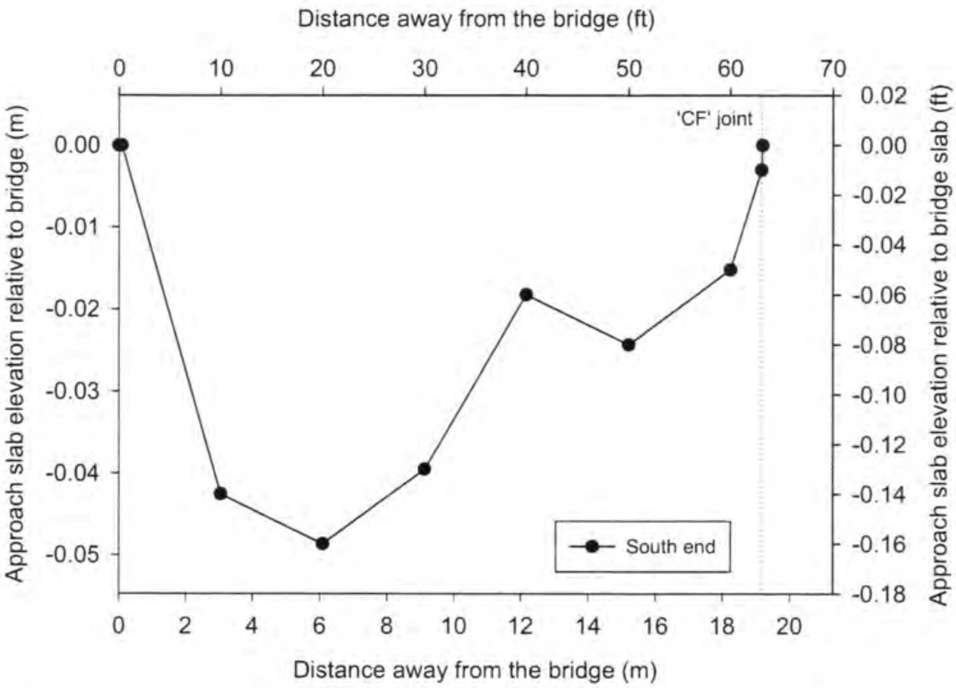


Figure 3.201 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7774.0L065 - South End of SBL)

Bridge No. 7777.0R065 (U.S. 65 over Vandalia Road/Railroad)

The bridge is a five span bridge with integral abutments and concrete girders. At the north end of the north bound lane, 10.2 cm differential settlement was measured (See Figure 3.202). A 15.2 cm void developed under the approach slab as shown in Figure 3.203. Flexible foam was used as a filler material of the expansion joint, which did not seal the expansion joint and gaps were observed. Moreover, concrete spalling at the expansion joint was observed. Although the embankment under the bridge had a gravel slope protection cover, erosion was observed between the abutment and the bridge embankment.

At the south end of the north bound lane, a 25.4 cm void was observed under the approach slab (See Figure 3.204). Figure 3.205 shows a 3.8 cm gap due to settlement of the approach slab relative to the wingwall. The expansion joint was not sealed where flexible foam was used as filler. Moreover, the sealer at the expansion joint was deteriorated as shown in Figure 3.206. Spalling of concrete at the expansion joint was also observed. Furthermore, erosion along the abutment sides was noted (See Figure 3.207). The embankment had a concrete slope protection for erosion control which was in a good condition. The embankment settled 17.8 cm (See Figure 3.208) and the abutment moved laterally 2.5 cm away from the embankment (See Figure 3.209).

Figure 3.202 - Differential Settlement at the Bridge Approach (Bridge No. 7777.0R065 - North End of NBL)

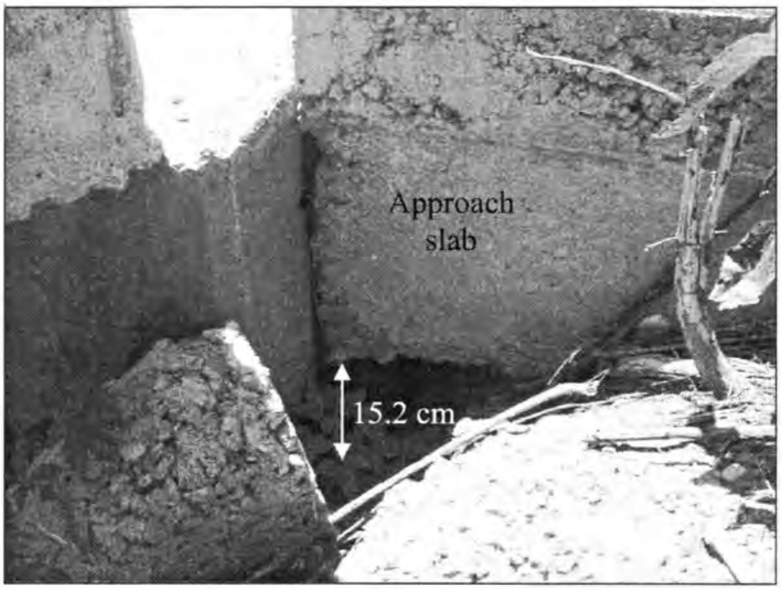


Figure 3.203 - A Void Developed under the Approach Slab (Bridge No. 7777.0R065 - North End of NBL)

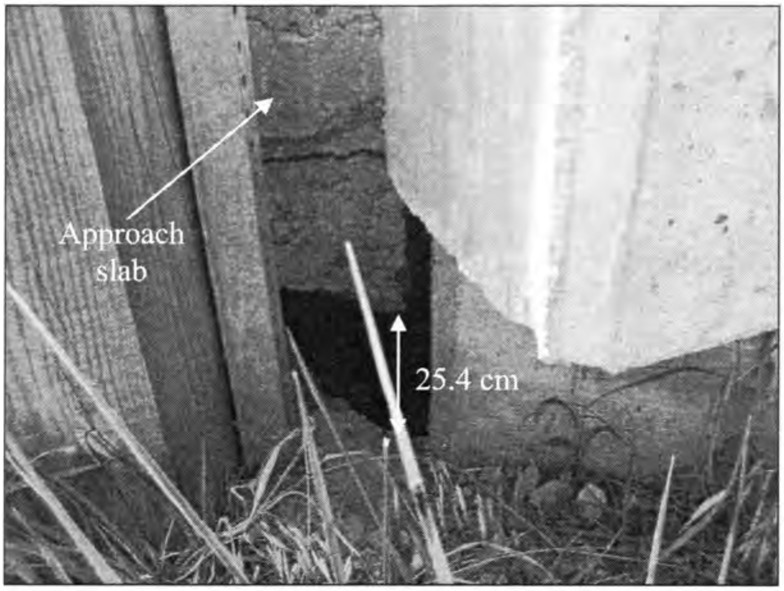


Figure 3.204 - A Void Developed under the Approach Slab (Bridge No. 7777.0R065 - South End of NBL)

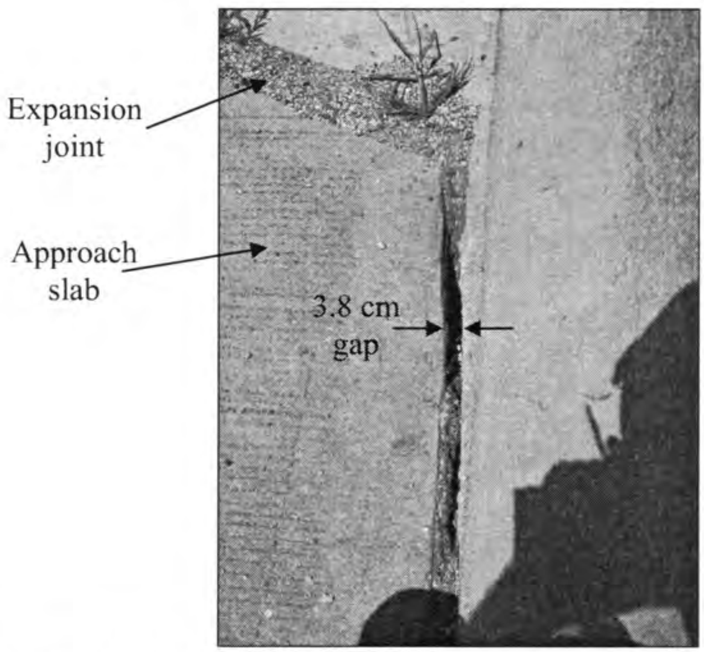


Figure 3.205 - A Gap Formed between the Approach Slab and the Wingwall (Bridge No. 7777.0R065 - South End of NBL)

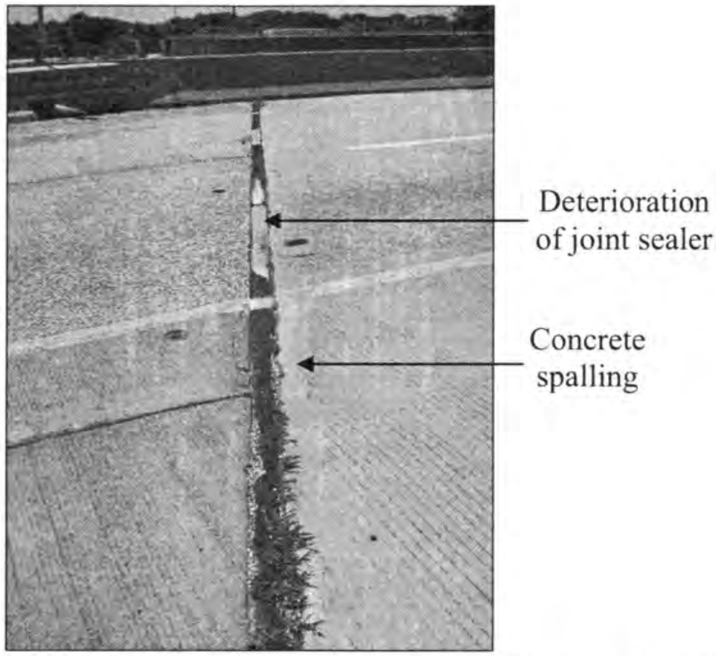


Figure 3.206 - Deteriorated Joint Sealer (Bridge No. 7777.0R065 - South End of NBL)

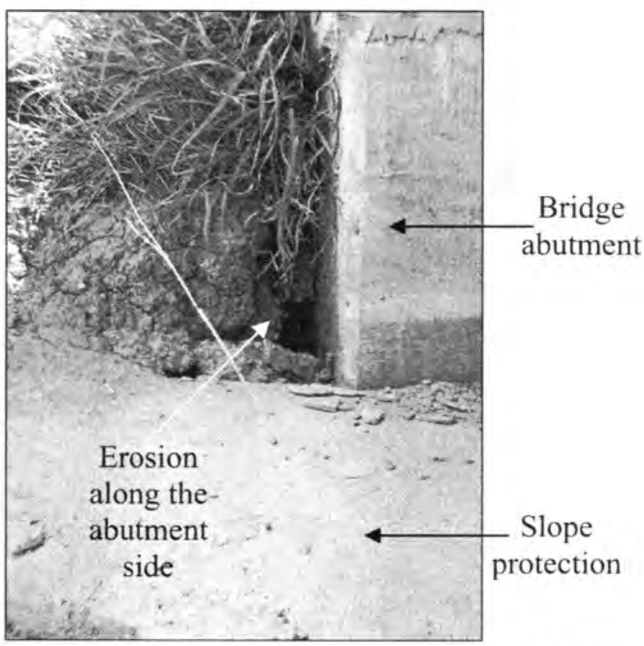


Figure 3.207 - Erosion Along the Abutment Sides (Bridge No. 7777.0R065 - South End of NBL)

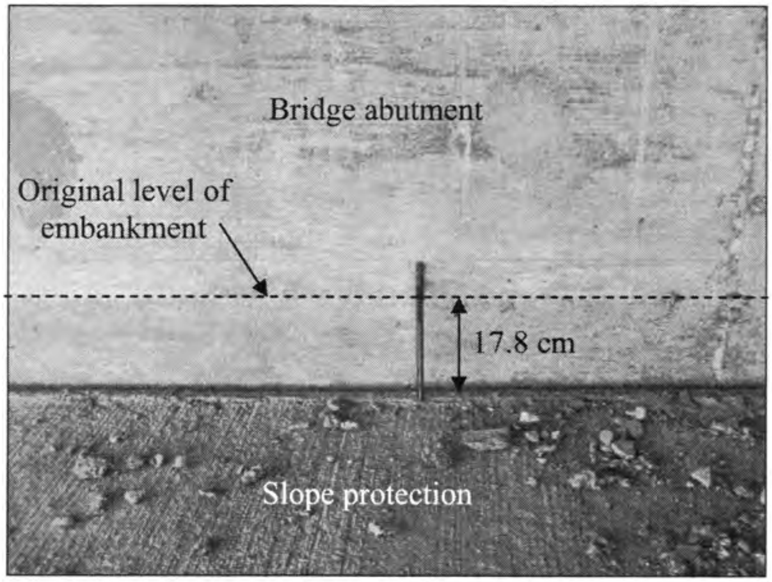


Figure 3.208 - Settlement of the Bridge Embankment (Bridge No. 7777.0R065 - South End of NBL)

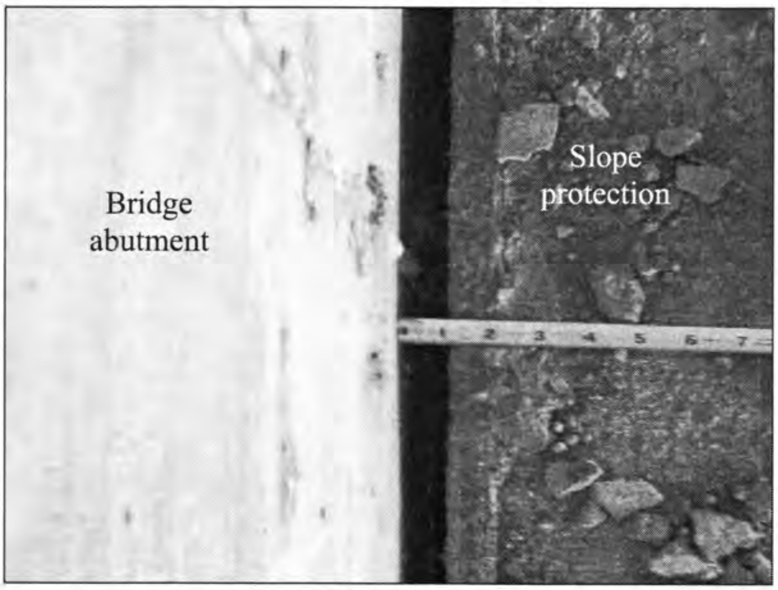


Figure 3.209 - Top View Showing Lateral Movement of the Abutment Away from the Bridge Embankment (Bridge No. 7777.0R065 - South End of NBL)

Table 3.4 shows the sizes of the voids measured under the approach slabs using approach slab coring. Approach slabs coring at both bounds of the bridge indicated a minimum and a maximum void size of 15.2 cm and 29.2 cm, respectively.

Bridge approach profiles were obtained for both ends of the north bound lane as shown in Figure 3.210. When compared with the assumed original profiles, the differential settlement at the north end was 3.8 cm at 6 m away from the bridge and 1.5 cm between the approach slab and the roadway. The maximum settlement of the south end approach slab was 3.3 cm at 12 m away from the bridge. Both profiles are sloping away from the bridge which indicates either compression of the embankment material or settlement of the foundation soil.

Table 3.4 - Voids Measured under the Approach Slabs

Bound	Void measured (cm)			
	North end		South end	
	Edge line	Center line	Edge line	Center line
North	20.3	25.4	25.4	24.1
South	21.6	21.6	29.2	15.2

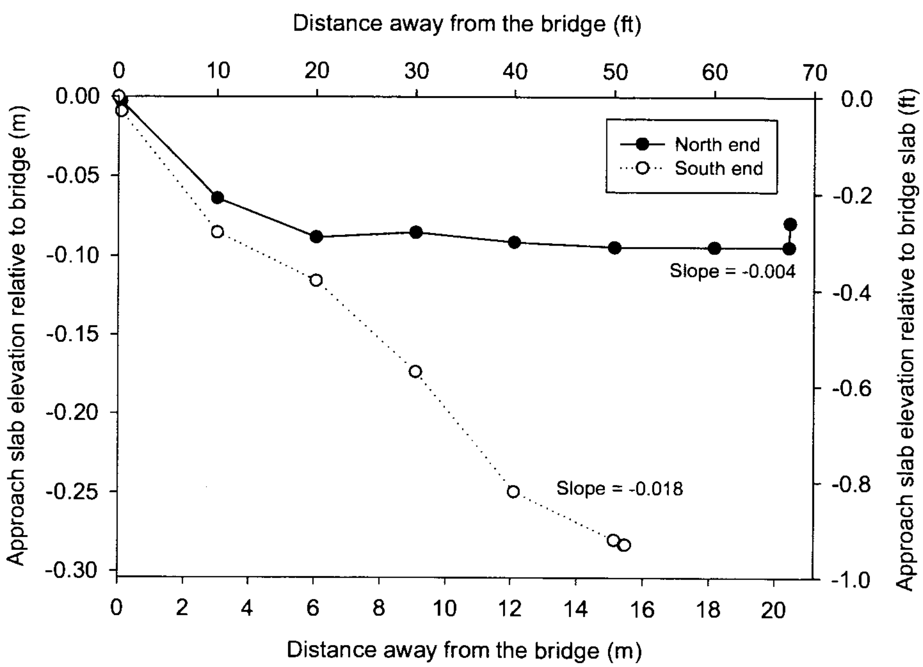


Figure 3.210 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7777.0R065 - NBL)

The bridge, which was also inspected in September 2004, was undergoing major maintenance. During inspection, replacement of the right lane approach slabs at the south bound lane was in progress. However, the left lane was still opened to traffic. The north end approach slab was removed while the south end approach slab was torn down and being removed (See Figure 3.211).

A void was observed under the north end of the left lane as shown in Figure 3.212. The void was 22.9 cm at the abutment and 2.5 cm at 1.2 m away from the bridge. Figure 3.213 shows the shear failure of concrete segments of the paving notch of the left lane where the approach slab was resting on only1.3 cm of the paving notch (See Figure 3.214). Refer to Chetlur (2004) for an analysis of the paving notch. This may be a result of having a narrow paving notch and/or movement of the abutment relative to the bridge approach. Therefore, tying the approach slab to the bridge abutment would prevent this problem from occurring.

At the south end approach slab, 2.5 cm differential settlement was observed (See Figure 3.215). Recycled tires were used as a joint filler material at the expansion joint. Similar to the north end, the approach slab at the south end was resting on 2.5 cm of the paving notch as shown in Figure 3.216. The figure also shows the void developed under the approach slab which was 25.4 cm deep at the abutment and extended 1.5 m away from the bridge.

DCP tests were performed at both the north and south ends of the south bound lane at approximately 0.6 m from the abutments. At the north end, the DCP test was conducted on the old special backfill material. At the south end, the DCP test was conducted on the new compacted special backfill (See Figure 3.217). The results of the DCP testing conducted on the old backfill material showed that the CBR values ranged from 2 to 15 as shown in Figure 3.218. DCP test results conducted on the new compacted special backfill showed that CBR values ranged from 1.5 to 15 (See Figure 3.219). There is no significant difference between the CBR values of both the old and the new compacted special backfill.

(a) North End Approach Slab

(b) South End Approach Slab

Figure 3.211 - Replacement of Approach Slabs (Bridge No. 7777.0R065 - SBL)

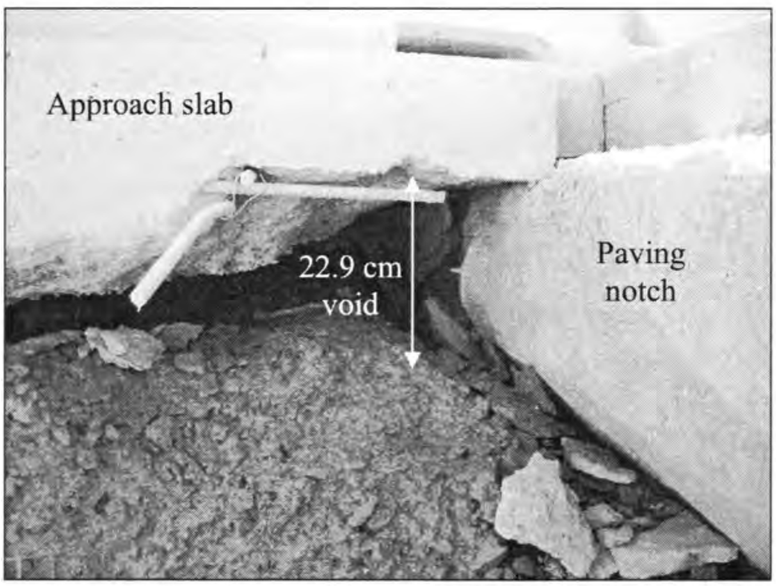


Figure 3.212 - A Void Developed under the Left Lane Approach Slab (Bridge No. 7777.0R065 - North End of SBL)

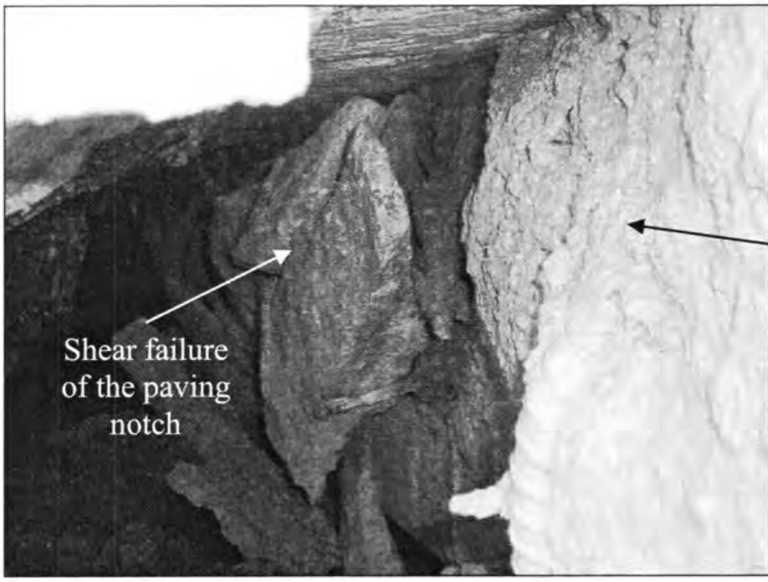


Figure 3.213 - Shearing of Paving Notch (Bridge No. 7777.0R065 - North End of SBL)

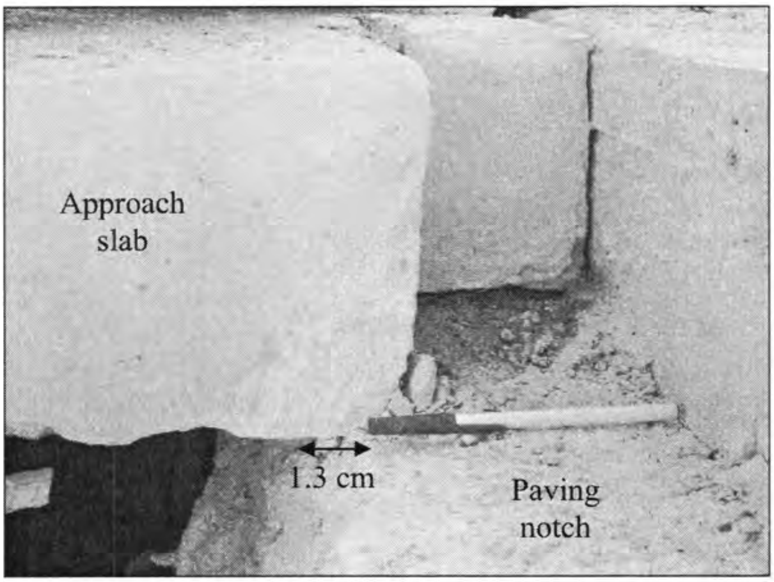


Figure 3.214 - The Approach Slab Resting on 1.3 cm of the Paving Notch (Bridge No. 7777.0R065 - North End of SBL)

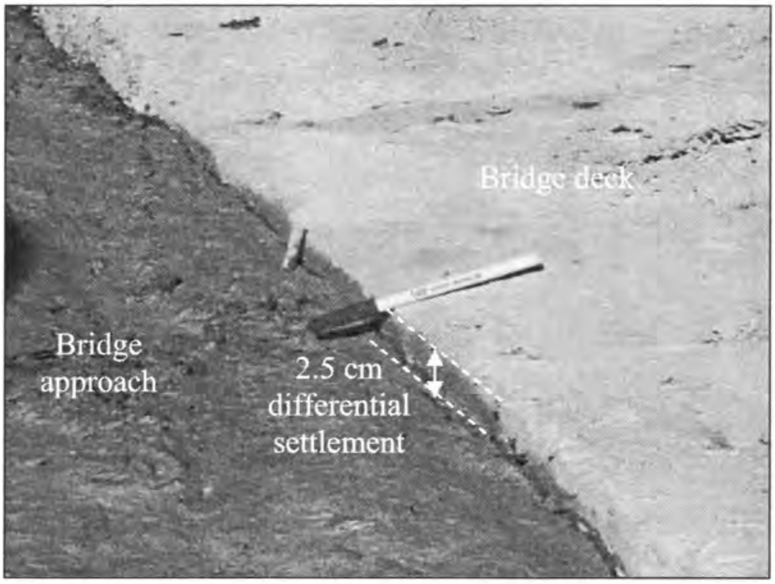


Figure 3.215 - Differential Settlement between the Bridge Deck and the Bridge Approach (Bridge No. 7777.0R065 - South End of SBL)

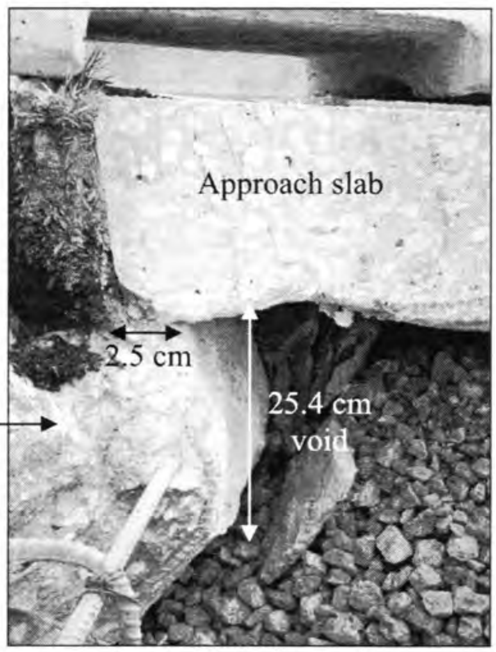


Figure 3.216 - Approach Slab Resting on 2.5 cm of the Paving notch, and a 25.4 cm Void Developed under the Approach Slab (Bridge No. 7777.0R065 - South End of SBL)

Figure 3.217 - Compacted sSecial Backfill (Bridge No. 7777.0R065 - South End of SBL)

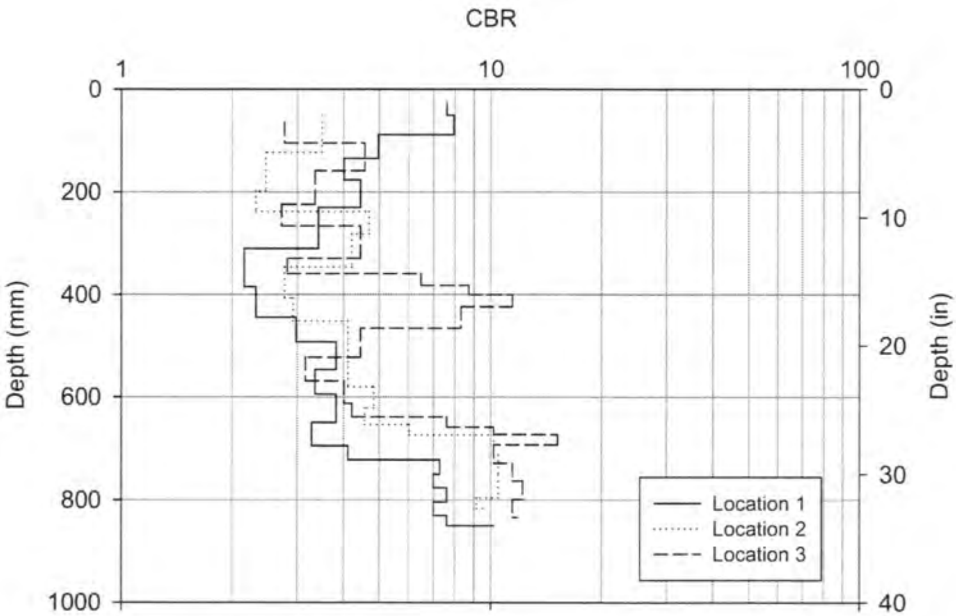


Figure 3.218 - Results of DCP Tests Conducted on the Old Backfill Material (Bridge No. 7777.0R065 - North End of SBL)

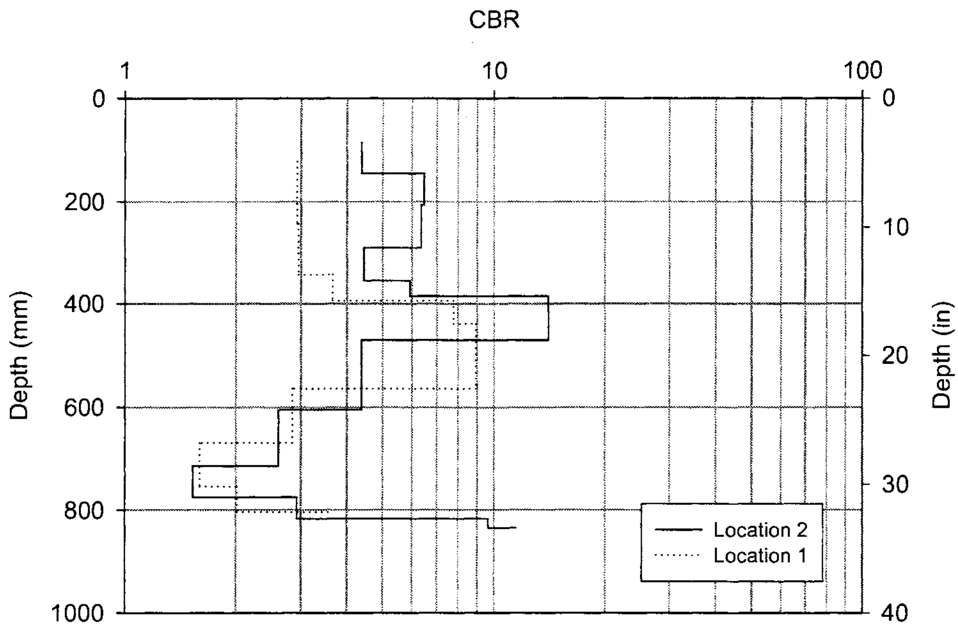


Figure 3.219 - DCP Tests Conducted on the New Compacted Special Backfill (Bridge No. 7777.0R065 - South End of SBL)

Bridge No. 7778.1065 (U.S. 65 over SE 6th Ave.)

The bridge is a three span bridge with non-integral abutments and steel girders. At the south end of the north bound lane, differential settlement was observed. The strip seal at the expansion joint was cut short causing water to flow around the bridge sides causing erosion along the abutment sides. The embankment had a slope protection which appeared to be in good condition with no settlement. At the north end of the north bound lane, erosion along the abutment sides was also observed. The bridge embankment settled a distance of 7.6 cm, as shown in Figure 3.220. The bridge embankment had a slope protection which appeared to be in good condition.

At the north end of the south bound lane, the approach slab settled a distance of 5.1 cm relative to the bridge deck, which was measured at the wingwall (See Figure 3.221). The outlet of the end drain could not be located.

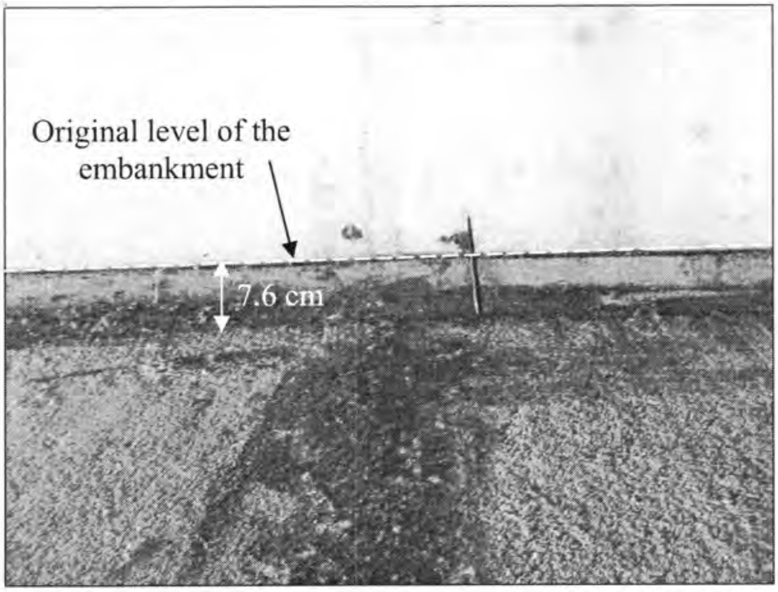


Figure 3.220 - Settlement of the Bridge Embankment (Bridge No. 7778.1065 - North End of NBL)

Figure 3. 221 - Differential Settlement of the Approach Slab (Bridge No. 7778.1065 - North End of SBL)

Figures 3.222 and 3.223 show the measured void size at the north end of the south bound lane. Table 3.5 summarizes the void sizes measured under the bridge approaches. The south end of the south bound lane was not cored because no significant settlement was observed at this end. The pavement thickness was 39. 4 cm which was determined by measuring the length of the core obtained at the edge line (See Figure 3.224).

The profiles of the north bound lane approach slabs were obtained as shown in Figures 3.225 and 3.226. Both the north and south approaches settled approximately 6.4 cm relative to the original slope at distances of 15 m and 18 m away from the bridge. The slopes of the original profiles are equal to 0.013 and -0.027 respectively (See Figure 3.225).

The north end of the south bound lane approach slab settled a distance of 3.8 cm relative to the roadway at the north end of the south bound lane, and settlement along the approach slab varied from 2.5 cm to 3.8 cm relative to the original slope, which has a slope of 0.017 (See Figure 3.226).

Table 3.5 - Voids Measured under Approach slabs

Bound	Void measured (cm)			
	North end		South end	
	Edge line	Center line	Edge line	Center line
North	0	10.2	1.3	1.3
South	7.6	8.9	-	-

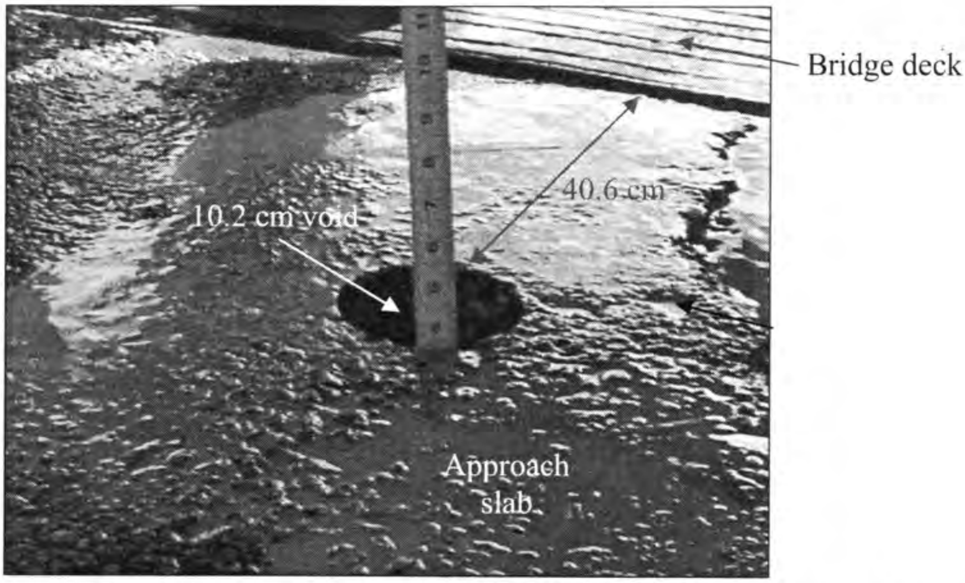


Figure 3.222 - A 10.2 cm Void under the Approach Slab Estimated by Measuring the Distance the Core Dropped (Bridge No. 7778.1065 - North End of SBL, Center Line)

Figure 3.223 - A 7.6 cm Void under the Approach Slab Estimated by Measuring the Distance the Core Dropped (Bridge No. 7778.1065 - North End of SBL, Edge Line)

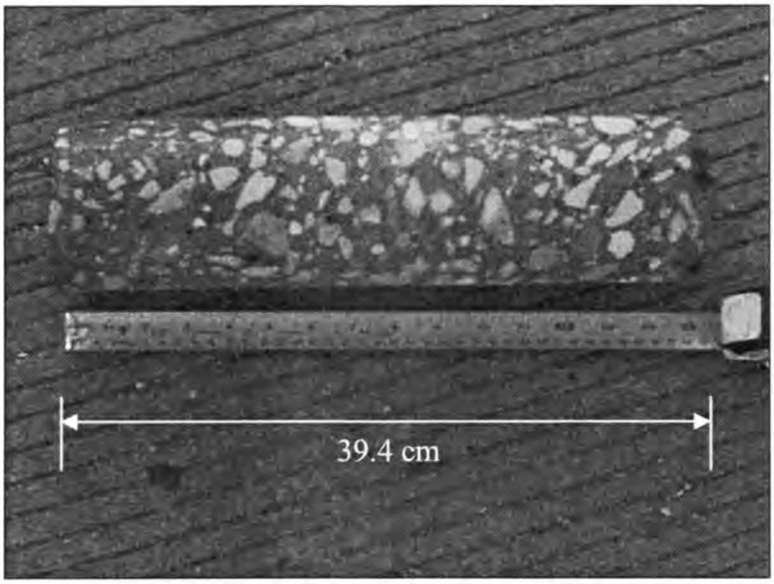


Figure 3.224 - Pavement Thickness Determined from Measuring the Length of the Core (Bridge No. 7778.1065 - North end of SBL, Edge Line)

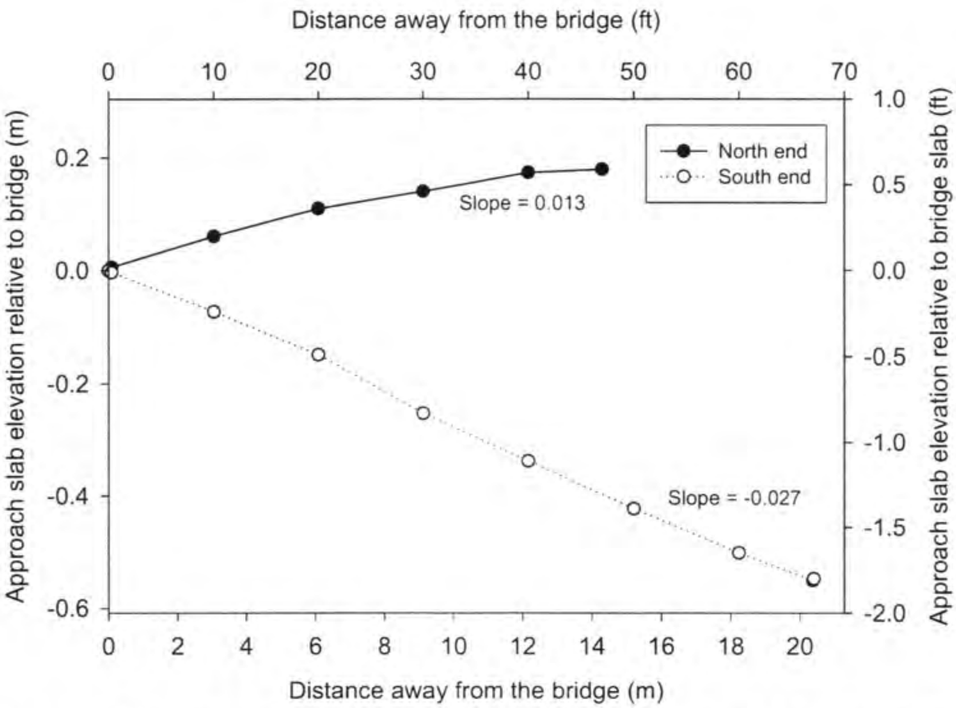


Figure 3.225 - Profile of the Bridge Approaches Relative to the Bridge Deck (Bridge No. 7778.1065 - NBL)

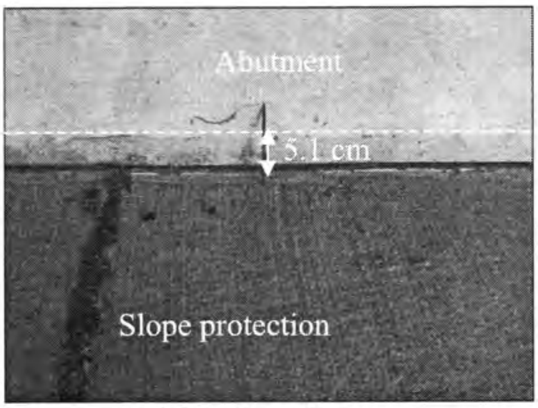
Figure 3.226 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7778.1065 - SBL)

<u>Bridge No. 7779.0065 (U.S. 65 over Rising Sun Dr.)</u>

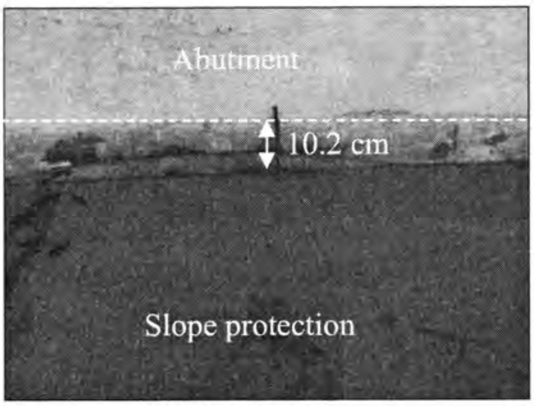
The bridge is a three span bridge with concrete girders and integral abutments. At the north end of the north bound lane, 3.8 cm differential settlement between the approach slab and the bridge deck was measured at the wingwall (See Figure 3.227). The width of the expansion joint was 14 cm and poorly sealed. The concrete slope protection of the embankment under the bridge was in good condition; however, the embankment settled a distance of 5.1 cm (See Figure 3.228).

At the south end of the north bound lane, erosion along the abutment side was observed. Furthermore, the embankment under the bridge settled 10.2 cm, and the abutment moved laterally 3.8 cm (See Figures 3.228 and 3.229).

Figure 3.227 - Differential Settlement between the Bridge Deck and the Bridge Approach (Bridge No. 7779.0065 - North End of NBL)



(a) North end



(b) South end

Figure 3.228 - Settlement of the Bridge Embankment (Bridge No. 7779.0065 - NBL)

Figure 3.229 - Lateral Movement of the Abutment Away from the Bridge Embankment forming a Gap (Bridge No. 7779.0065 - South End of NBL)

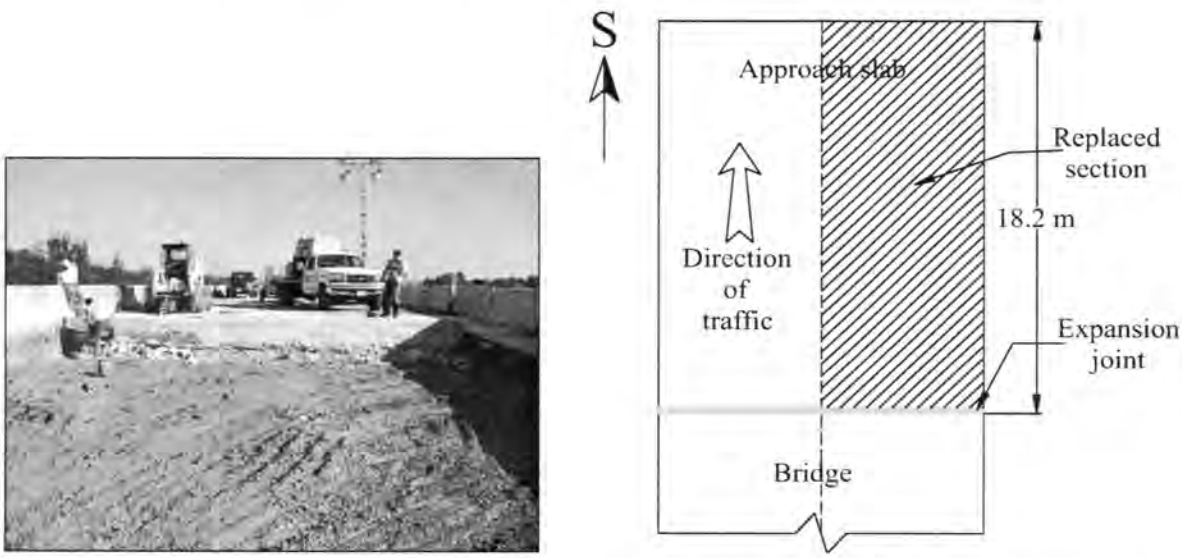
The snake camera was used to inspect the subdrains of both ends of the north bound lane. At the north end, the subdrain was completely dry with no indication of water or soil. At 1.5 m inside the subdrain, a rodent nest was observed blocking the subdrain (See Figure 3.230), which indicates that the subdrain is not functioning. At the south end, the subdrain was completely dry with some fines at the bottom of the drain (See Figure 3.231), which indicates that the subdrain used to function.

Figure 3.230 - Rodent Nest blocking the Subdrain at 1.5 m from the Outlet (Bridge No. 7779.0065)

Figure 3.231 - Snake Camera Inserted Inside the Subdrain (Bridge no. 7779.0065)

The bridge was also inspected in September 2004 during which the approach slab at the south end of the south bound lane was being replaced. The replaced section was on the right lane, while the left lane approach slab was still open to traffic (See Figure 3.232). Figure 3.233 shows, the tearing of the old pavement notch which was 20.3 cm wide. The new paving notch will be 38.1 cm wide. The void under the existing approach (left lane) was approximately 0.5 m deep at the abutment and 15.2 cm deep at 0.9 m away from the abutment (See Figure 3.234). A new subdrain pipe was installed behind the bridge abutment. The subdrain was connected to the end drain. Figure 3.235 shows the trenches excavated to place the subdrain and the end drain. Porous backfill was used around the new installed drains.

DCP tests were conducted at 3 locations at approximately 0.6 m away from the abutment. The test was conducted twice- before excavating the old special backfill and after placing the new special backfill. The results which are shown in Figures 3.236 and 3.237 illustrate that prior to excavation, the old special backfill was not compacted to the required density, and CBR values ranged from 2 to 9 (See Figure 3.236), while the CBR values for the newly placed special backfill ranged from 3 to 18 which indicate that this special backfill was not compacted (See Figure 3.237); and therefore, this backfill is expected to settle causing differential settlement and void development as observed previously.



(a) Replaced Approach Slab **(b) Schematic of the Replaced Section**

Figure 3.232 - Replacement of the Right Lane Approach Slab (Bridge No. 7779.0065 - South End of SBL)

Figure 3.233 - Tearing Down the Old Pavement Notch (Bridge No. 7779.0065 - South End of SBL)

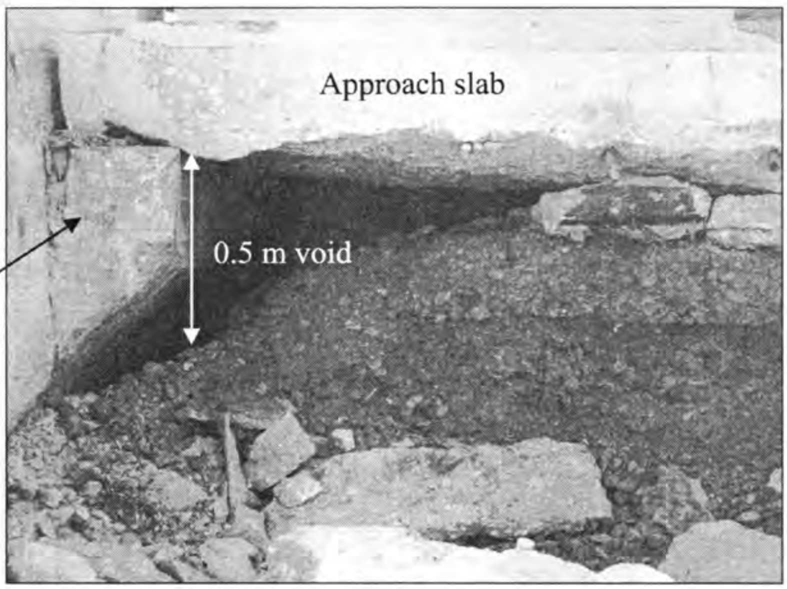


Figure 3. 234. A Void Developed under the Left Lane Approach Slab (Bridge No. 7779.0065 - South End of SBL)

(a) Trench Excavated to Install Subdrain

(b) Trench Excavated to Connect the Subdrain to the End Drain

Figure 3.235 - Installing a Drainage System under the Right Lane Approach Slab (Bridge No. 7779.0065)

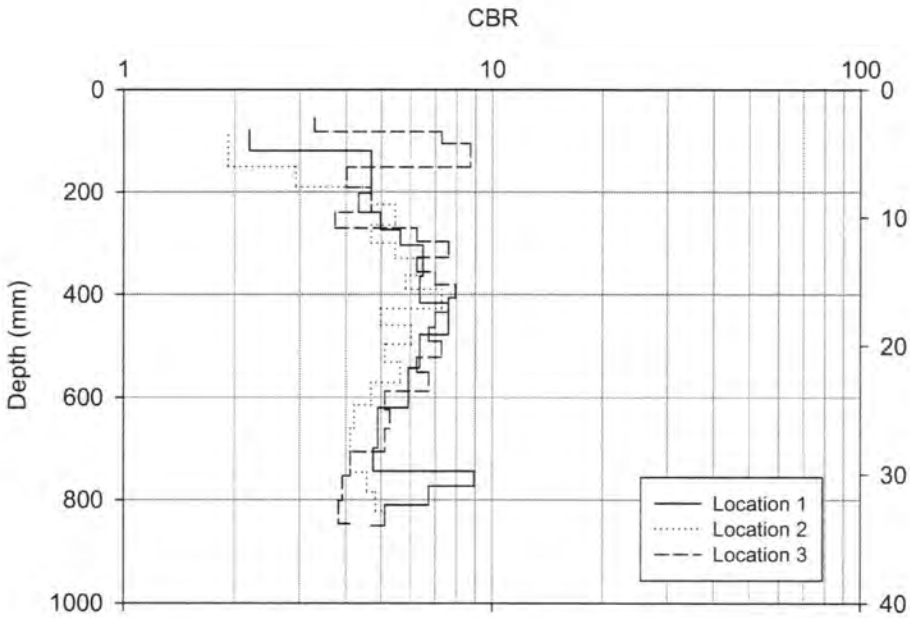


Figure 3.236 - Results of DCP Test Conducted on Special Backfill Material before Excavation (Bridge No. 7779.0065 - South End of SBL)

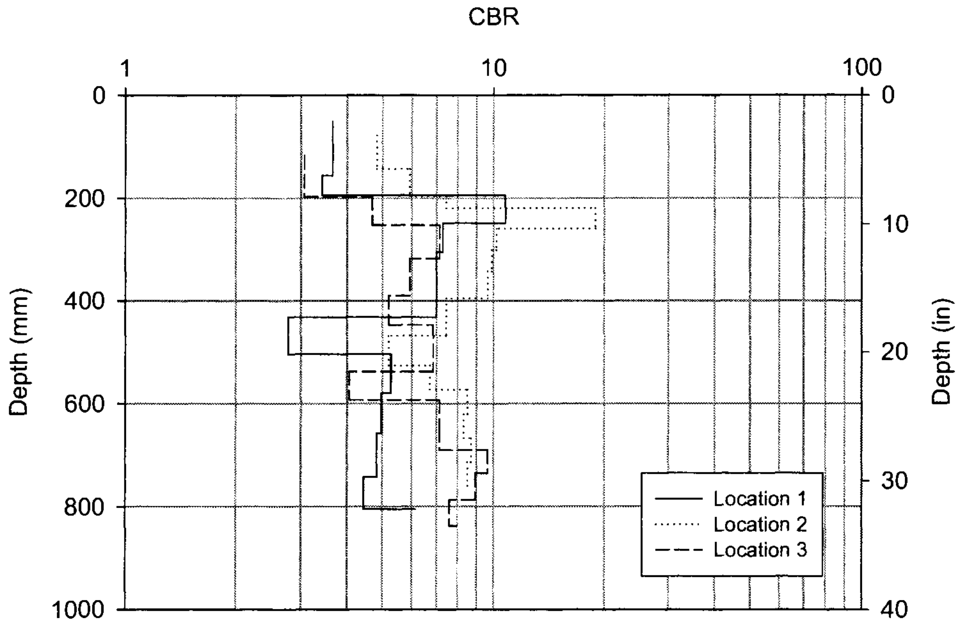


Figure 3.237 - Results of DCP Test Conducted on Special Backfill Material after Replacement (Bridge No. 7779.0065 - South End of SBL)

<u>Bridge No. 7779.4065 (U.S. 65 over IA 163)</u>

The bridge is a four span bridge with concrete girders and integral abutments. At the north end of the north bound lane, the approach had an asphalt overlay which was cracked and experienced differential settlement of 11.4 cm, which was measured relative to the wingwall. Erosion along the sides of the abutment was observed. The embankment had a concrete slope protection that was in a good condition. However, the embankment settled a distance of 8.9 cm. The outlet of the end drain was in a satisfactory condition.

At the south end of the north bound lane, differential settlement of 6.4 cm was measured relative to the wingwall (See Figure 3.238). Concrete spalling and cracking of the asphalt overlay were observed at the expansion joint (See Figure 3.239). Concrete slope protection of the bridge embankment was in a good condition; however, it settled 8.9 cm (See Figure 3.240), and the abutment moved laterally 1.3 cm

At the north end of the south bound lane, differential settlement was 10.2 cm (See Figure 3.241). Recycled tires were used as a joint filler of the expansion joint as shown in Figure 3.242. The width of the expansion joint was 12.7 cm. The concrete slope protection was in a good condition with no cracks visible.

The approach slabs at both ends of the south bound lane were cored. Table 3.6 shows the measured void size, pavement thickness, and paving notch dimension at the approach slabs of both ends. The void under the approach slab was largest (i.e., 24.1 cm) at the edge line of the north end approach slab (See Figure 3.243).

Profiles were obtained for all four ends of the bridge (See Figures 3.244 and 3.245). The differential settlement of the approach slab relative to the bridge deck for both ends of the north bound lane was 2.5 cm. At the south bound lane, the maximum settlement relative to the original profile was 5.1 cm and 4.6 cm for the north and south end approach slabs, respectively. The slopes of the north and south ends of the north bound lane are -0.013 and -0.001 respectively, and -0.015 and -0.002 for the north and south ends of the south bound lane. Both approach slabs at the south bound lane are sloping away from the bridge, which indicates either settlement of the foundation soil or compression of the embankment material.

Figure 3.238 - Differential Settlement (Bridge No. 7779.4065 - South End of NBL)

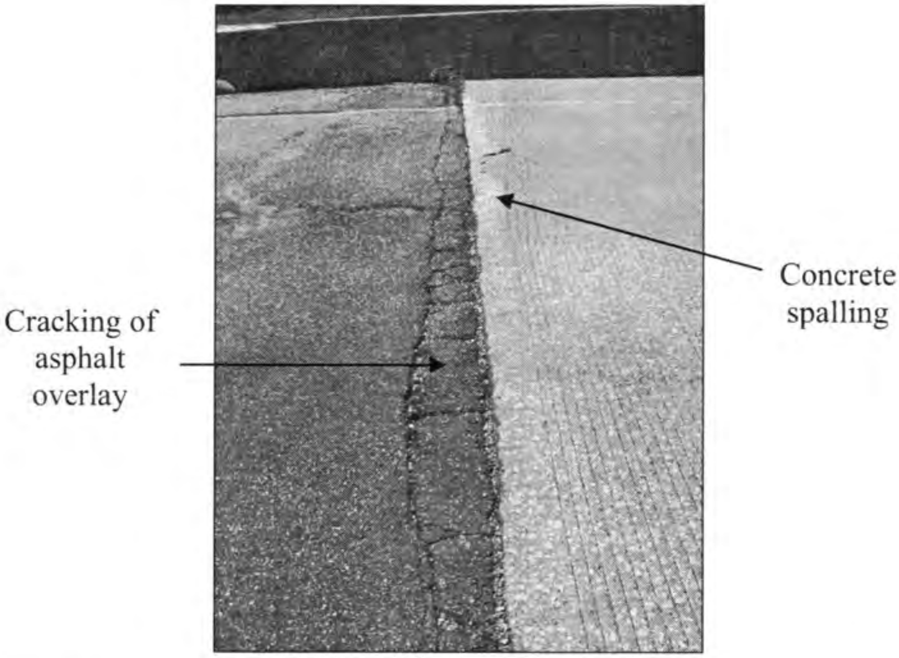


Figure 3.239 - Concrete Spalling and Cracking of Asphalt Overlay at the Expansion Joint (Bridge No. 7779.4065 - South End of NBL)

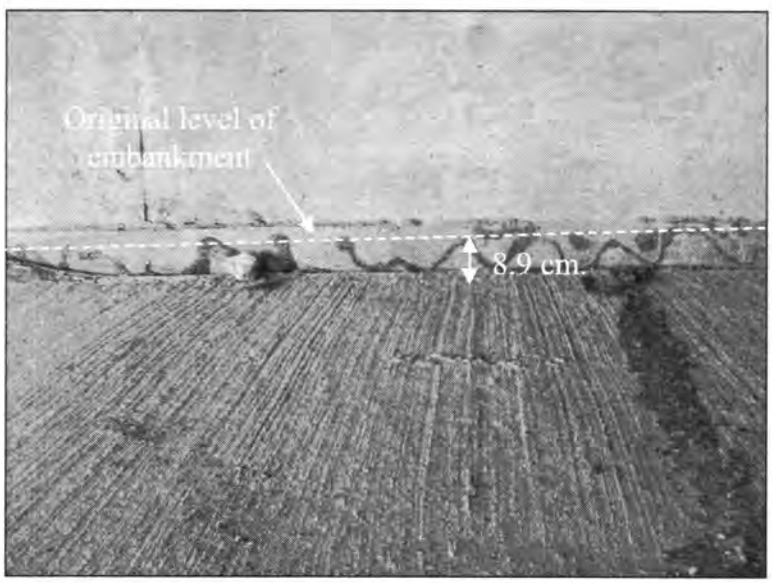


Figure 3.240 - Settlement of the Bridge Embankment (Bridge No. 7779.4065 - South End of NBL)

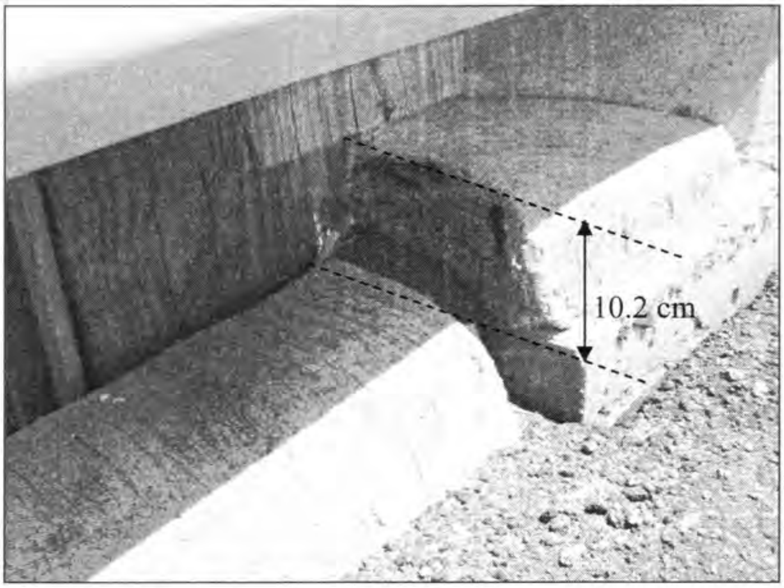


Figure 3.241 - Differential Settlement at the Bridge Approach (Bridge No. 7779.4065 - North End of SBL)

Figure 3.242 - Recycled Tires used as Joint Filler (Bridge No. 7779.4065)

Table 3.6 - Measurements of the Void Size, Paving Notch, and Pavement Thickness (SBL)

Bridge End	Void size (cm)		Notch dimension (cm)		Pavement thickness (cm)	
	Edge line	**Center line**	**Edge line**	**Center line**	**Edge line**	**Center line**
North	9.5	6.5	-	10.25	-	-
South	5.0	3.75	8.5	9.0	12.5	14

Figure 3.243 - Void under the Approach Slab Estimated by Measuring the Distance the Core Dropped (Bridge No. 7779.4065 - North End of SBL)

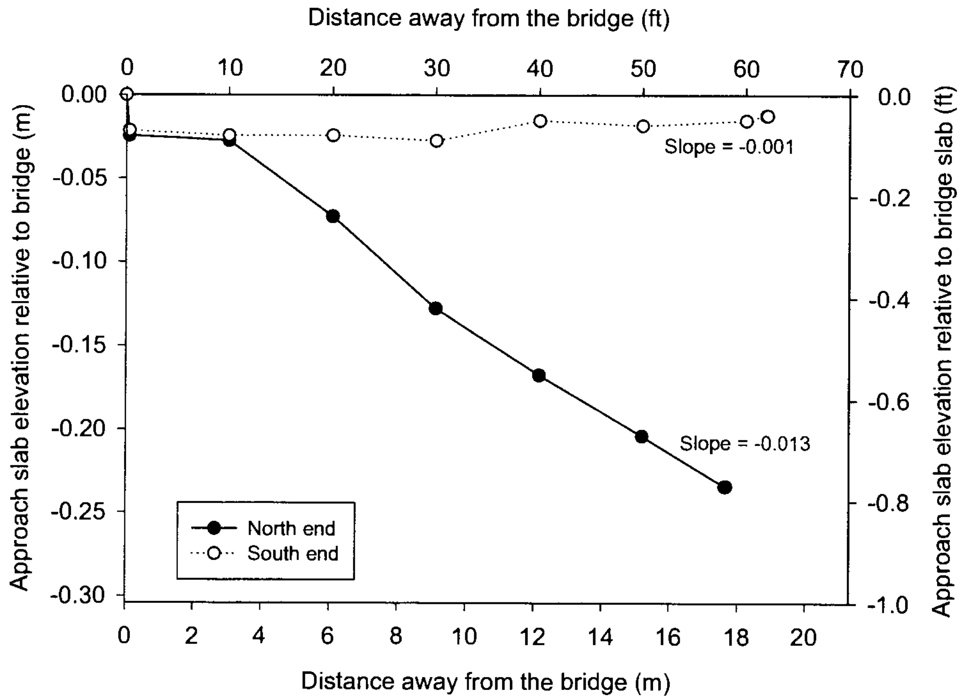


Figure 3.244 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7779.4065 - NBL)

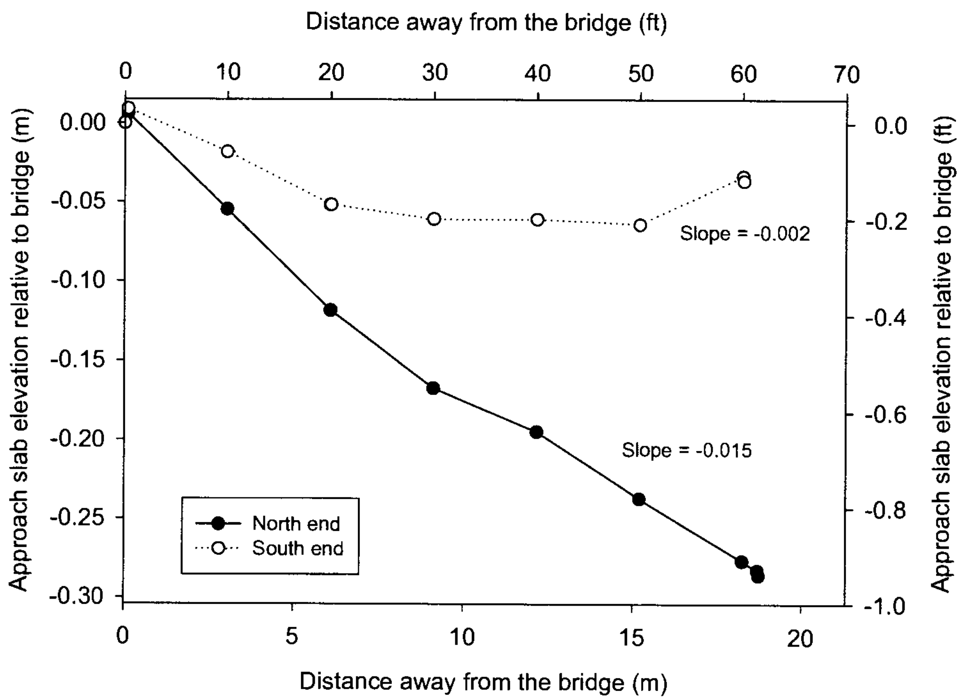


Figure 3.245 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7779.4065 - SBL)

Although, the ISU research team attempted to use the snake camera at the north end of the north bound lane subdrain outlet, the subdrain was blocked with soil as show in Figure 3.246. The drain was filled with soil. Unlike other bridges where the subdrain outlet was located beside the abutment, this outlet was located at the bottom of the embankment.

Figure 3.246 - Subdrain Outlet, Located at the Bottom of the Embankment, Blocked by Soil Particles (Bridge no. 7779.4065)

<u>Bridge No. 80.8R065 (U.S. 65 over NE 27th St.)</u>

The bridge is a three span bridge with concrete girders and integral abutments. At the north end of the north bound lane, differential settlement was 6.4 cm, which was measured relative to the wingwall. Transverse cracks at the approach slab were visible (See Figure 3.247). Flexible foam was used as a filler at the expansion joint which was 8.3 cm wide and poorly sealed allowing water to flow around the bridge. Erosion along the abutment sides was observed. No settlement was observed at the embankment under the bridge and the concrete slope protection appeared to be in a good condition. The outlet of the subdrain was in a satisfactory condition but appeared to be dry with no indication of water flowing out.

Approach slab Coring was performed at the right lane of the north end of the north bound lane. The measured void under the approach slab was 3.8 cm at the edge line and 5.1 cm at the center line.

The profile of the north end was obtained (See Figure 3.248). The maximum settlement relative to the original profile, which has a slope of 0.016, was approximately 5.1 cm at 6.1 m away from the bridge.

Figure 3.247 - Transverse Cracks at the Bridge Approach (Bridge No. 80.8R065)

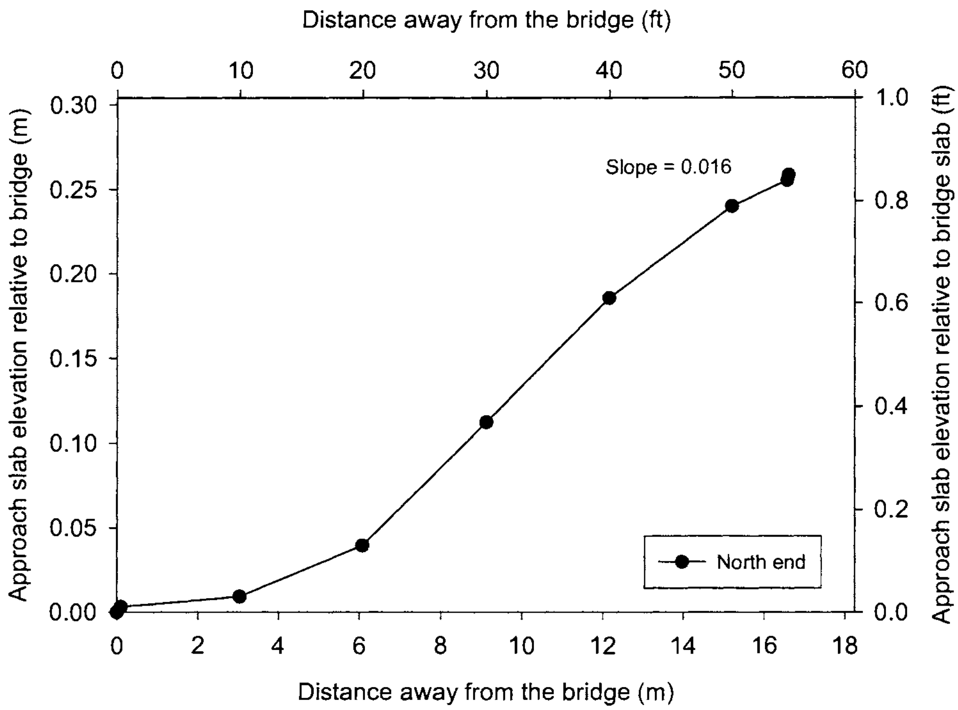


Figure 3.248 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 80.8R065 - North End of NBL)

<u>Bridge No. 7781.2065 (U.S. 65 over 4 Mile Creek/ Railroad)</u>

The bridge is a four span bridge with steel girders and non-integral abutments. At the north end of the south bound lane, differential settlement was 7.6 cm relative to the wingwall. Significant damage to the approach pavement was observed (See Figure 3.249). Although, the expansion joint condition was satisfactory, the strip seal was cut short allowing water to run down the bridge resulting in erosion around the abutment. At the embankment under the bridge, no slope protection was used, therefore erosion was noticeable. The outlet of the end drain was damaged, and the end drain appeared to be dry and not functioning (See Figure 3.250).

At the south end an asphalt overlay was placed to alleviate the differential settlement at the bridge approach. Nonetheless, differential settlement of 5.1 cm relative to the wingwall was observed (See Figures 3.251 and 3.252). The end drain appeared to be in a good condition.

Figure 3.253 shows the approach slabs coring at both bounds. Table 3.7 presents the void sizes measured under each approach slab. The maximum voids measured under the edge line at the north and south end approach slabs were 11.4 cm and 12.7 cm, respectively. The void under the north end of the south bound lane approach slab was 1.3 cm as shown in Figure 3.254. As performed at other bridges expansive foam was placed after obtaining cores as shown in Figure 3.255 to block the hole formed due to coring.

Figure 3.249 - Severe Cracking at the Bridge Approach Pavement (Bridge No. 7781.2065 - North End of NBL)

Figure 3.250 - Damaged End Drain Outlet (Bridge No. 7781.2065 - North End of NBL)

Figure 3.251 - Settlement of Bridge Approach (Bridge No. 7781.2065 - South End of NBL)

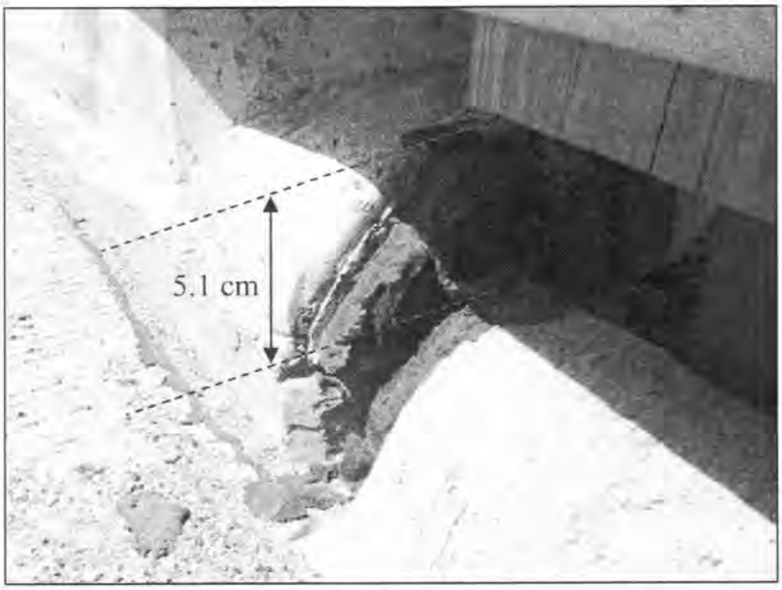


Figure 3.252 - Differential Settlement at the Bridge Approach (Bridge No. 7781.2065 - South End of NBL)

Table 3.7 - Voids Measured the under Approach Slab

Bound	Void measured (cm)			
	North end		South end	
	Edge line	Center line	Edge line	Center line
North	11.4	2.5	2.5	2.5
South	1.3	7.6	12.7	11.4

Figures 3.256 and 3.257 show the bridge approach profiles of all four ends. The approach slab at the north end of the north bound lane settled 5.1 cm at 6 m away from the bridge. The south end approach slab settled 10.2 cm at 6 m away from the bridge. The original slopes of the north and south ends of the north bound lane approach slabs are -0.036 and 0.0058 respectively.

At the south bound lane, the north end approach slab settled 2.5 cm at 12 m away from the bridge relative to the original profile, while the south end settled 12.7 cm at 6 m away from the bridge. The original slopes of the north and south ends of the south bound lane approach slabs are -0.029 and -0.001 respectively

Figure 3.253 - Coring at the Bridge Approach (Bridge No. 7781.2065 - NBL)

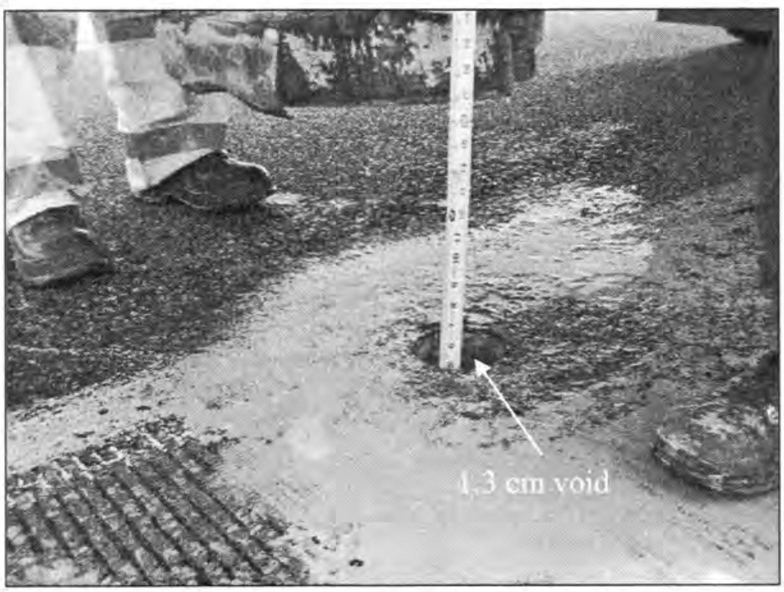


Figure 3.254 - Measuring the Void under the Approach Slab (Bridge No. 7781.2065 - North End of SBL)

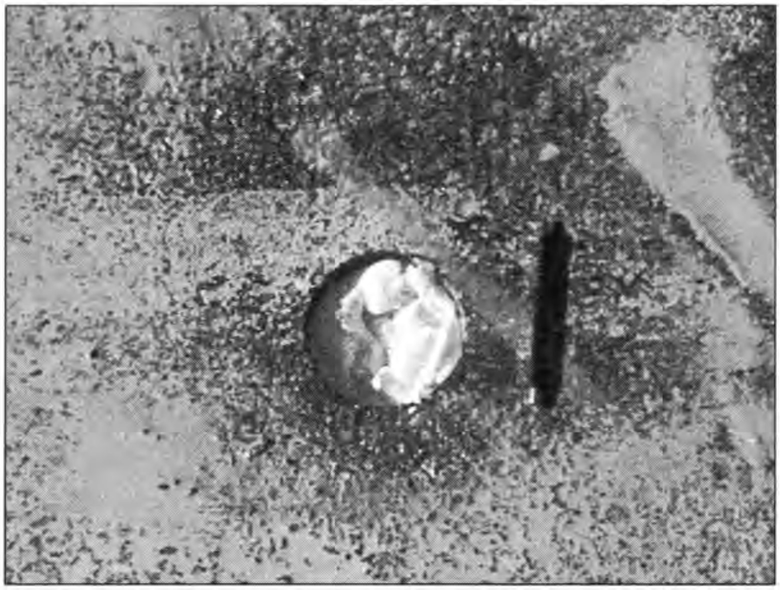


Figure 3.255 - Expanding Foam Placed in the Cored hole of the Approach Slab (Bridge No. 7781.2065 - NBL)

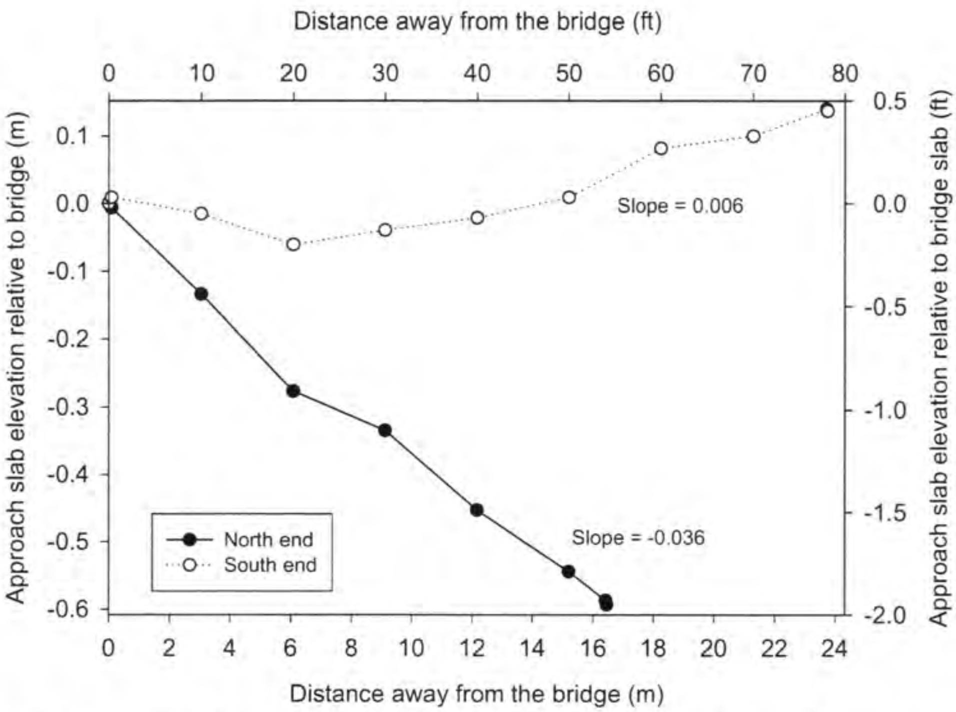


Figure 3.256 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7781.2065 - NBL)

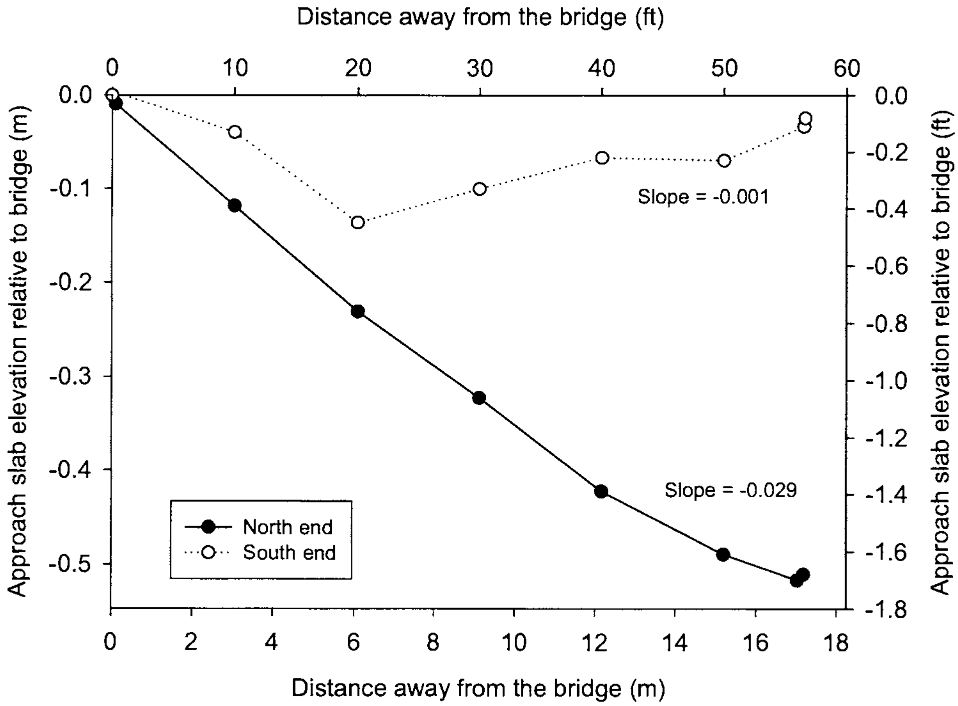


Figure 3.257 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7781.2065 - SBL)

<u>Bridge No. 7782.8L065 (U.S. 65 over NE 46th Ave.)</u>

The bridge is a three span bridge with steel girders and non-integral abutments. At the north end of the south bound lane, differential settlement at the approach slab was observed (See Figure 3.258). Transverse cracking across the approach slab was also noted (See Figure 3.259). The expansion joint filler was not sealing the joint allowing water to flow around the bridge. The strip seal was also observed filled with soil particles and cut short. The concrete slope protection of the embankment was in a good condition.

At the south end of the south bound lane differential settlement was observed as shown in Figure 3.260. This differential settlement was 3.8 cm at the wingwall. The strip seal was filled with soil particles but the overall condition of the expansion joint was satisfactory. The concrete slope protection was in good condition with no observed settlement (See Figure 3.261).

Bridge approaches at both ends of the south bound lane were cored. Table 3.8 shows the measured void sizes at both ends. The voids under the approach slab of both ends were 5.1 cm at the edge line and 3.8 cm at the center line.

Profiles of both approach slabs at the edge line of south bound lane are shown in Figure 3.262. The maximum settlement of the north end approach slab relative to the original profile was 3 cm at 3 m away from the bridge. The maximum settlement of the south end approach slab relative to the original profile was 7.6 cm at 12 m away from the bridge. The slopes of the original profiles of both the north and south ends of the south bound lane are 0.009 and -0.026 respectively.

Figure 3.258 - Settlement of the Bridge Approach (Bridge No. 7782.8L065 - North End of SBL)

Figure 3.259 - Transverse Cracking at the Bridge Approach (Bridge No. 7782.8L065 - North End of SBL)

Figure 3.260 - Bridge Approach Settlement (Bridge No. 7782.8L065 - South End of SBL)

Figure 3.261 - Bridge Embankment in a Satisfactory Condition (Bridge No. 7782.8L065)

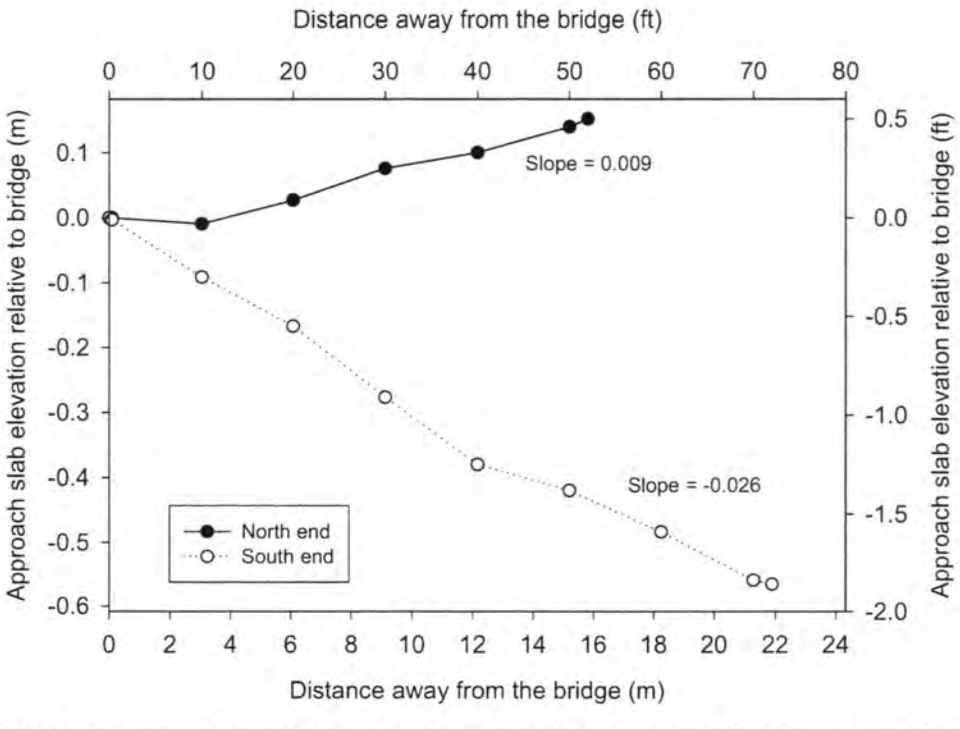


Figure 3.262 - Profile of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7782.8L065 - SBL)

Table 3.8 - Voids Measured under the Approach Slab

Bound	Void measured (cm)			
	North end		South end	
	Edge line	Center line	Edge line	Center line
South	5.1	3.8	5.1	3.8

Bridge No. 7783.1065 (U.S. 6/Hubbell)

The bridge is a four span bridge with concrete girders and integral abutments. Although an asphalt overlay was placed at the north end of the north bound lane, differential settlement of 5.1 cm was measured relative to the wingwall (See Figure 3.263). A 5.1 cm gap between the bridge approach and the wingwall developed due to settlement of the approach slab relative to the wingwall (See Figure 3.264). Recycled tires were used as a joint filler of the expansion joint. The embankment settled 10.2 cm and the abutment moved laterally 5.1 cm from the bridge embankment as shown in Figure 3.265. The slope protection however was still in a satisfactory condition.

At the south end of the north bound lane, differential settlement was 2.5 cm (See Figure 3.266). The width of the expansion joint was 12.7 cm. Recycled tire was used as a joint filler which did not properly seal the joint allowing water to flow around the bridge. Erosion along the abutment sides was observed. Settlement of the embankment was 10.2 cm Furthermore, the end drain was observed damaged and not functioning.

At the north end of the south bound lane differential settlement was 5.1 cm relative to the wingwall. Erosion along the sides of the abutment was noticed. The concrete slope protection of the embankment under the bridge was in a good condition; however, the embankment settled 7.6 cm and a 5.1 cm gap developed between the embankment and the abutment due to the lateral movement the bridge structure (See Figure 3.267).

Figure 3.263 - Asphalt Overlay Placed Over the Approach Slab to Compensate for the Differential Settlement (Bridge No. 7783.1065 - North of End NBL)

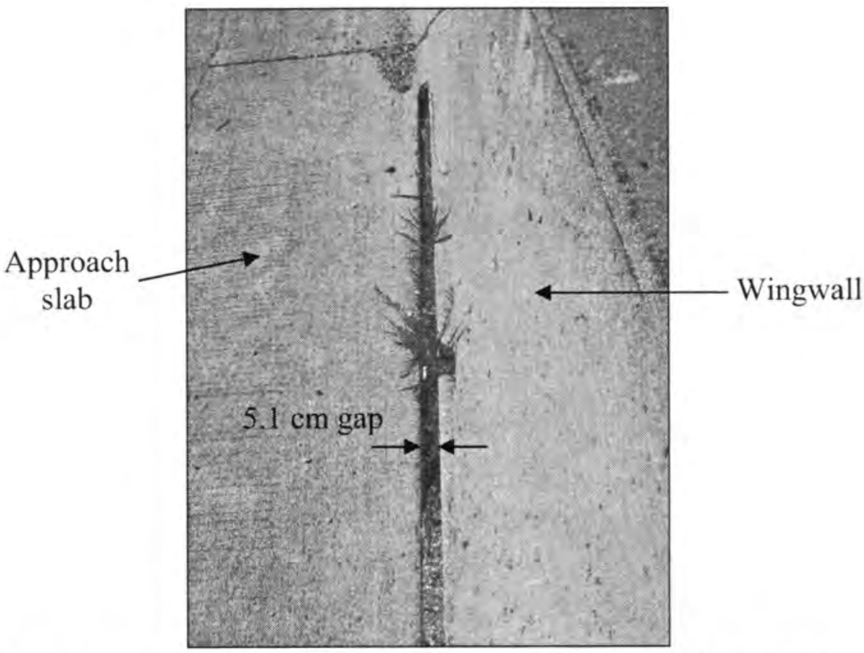


Figure 3.264 - A Gap Formed between the Approach Slab and the Wingwall Due to Settlement of the Approach Slab (Bridge No. 7783.1065 - North End of NBL)

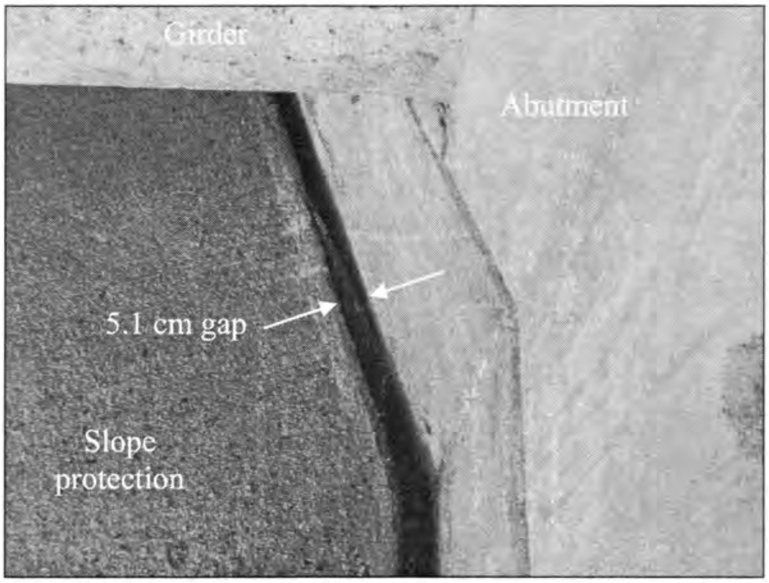


Figure 3.265 - Lateral Movement of the Abutment Away from the Bridge Embankment Caused by Expansion of the Bridge Structure (Bridge No. 7783.1065 - North End of NBL)

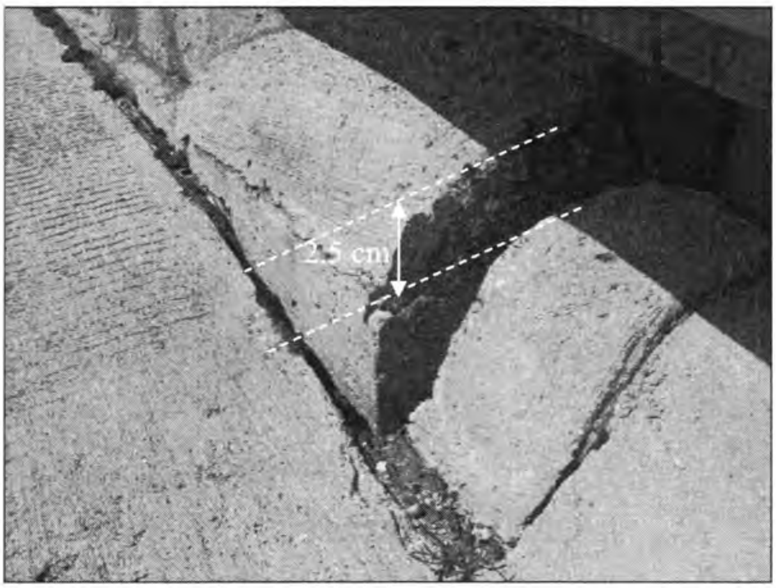


Figure 3.266 - Differential Settlement at the Bridge Approach (Bridge No. 7783.1065 - South End of NBL)

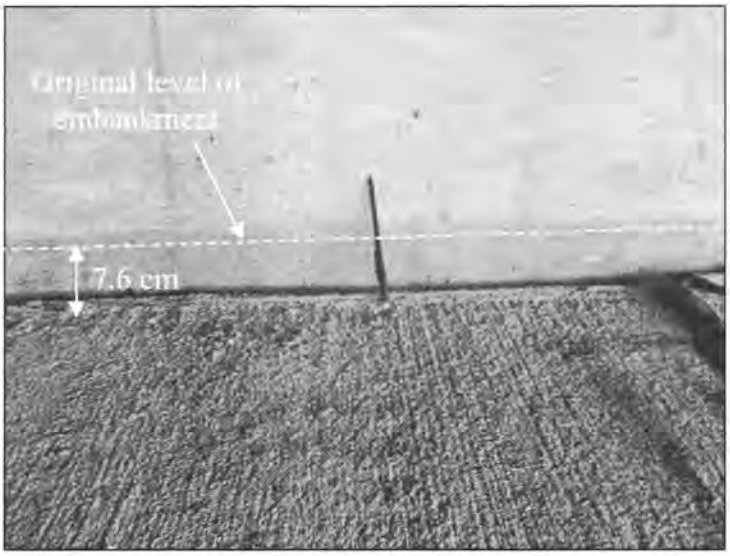


Figure 3.267 - Settlement of Embankment (Bridge No. 7783.1065 - North End of SBL)

Coring of the approach slabs at the north end indicated that the void was 16.5 cm at the edge line and 17.8 cm at the center line. At the south end, the measured void was 14 cm at both the edge line and center line.

The profiles of all four approach slabs of the bridge were obtained from the edge line of the right lane (Figures 3.268 and 3.269). At the north bound lane, differential settlement between the north end approach slab and the roadway was 1.3 cm, and the maximum settlement relative to the original profile was 3.8 cm at 12 m away from the bridge. At the south end, the maximum settlement relative to the original profile was 2.5 cm at 12 m away from the bridge. The slopes of the original profiles of both the north and south ends of the north bound lane are -0.012 and -0.009 respectively. Both profiles are sloping away from the bridge indicating either settlement of the foundation soil or compression of the embankment material.

At the south bound lane, the maximum settlement at the north end approach slab relative to the original profile was 3.8 cm at 10 m away from the bridge, while at the south end approach slab the maximum settlement was 7.6 cm at 6 m away from the bridge. The slopes of the original profiles of both the north and south ends of the south bound lane are -0.009 and -0.008 respectively.

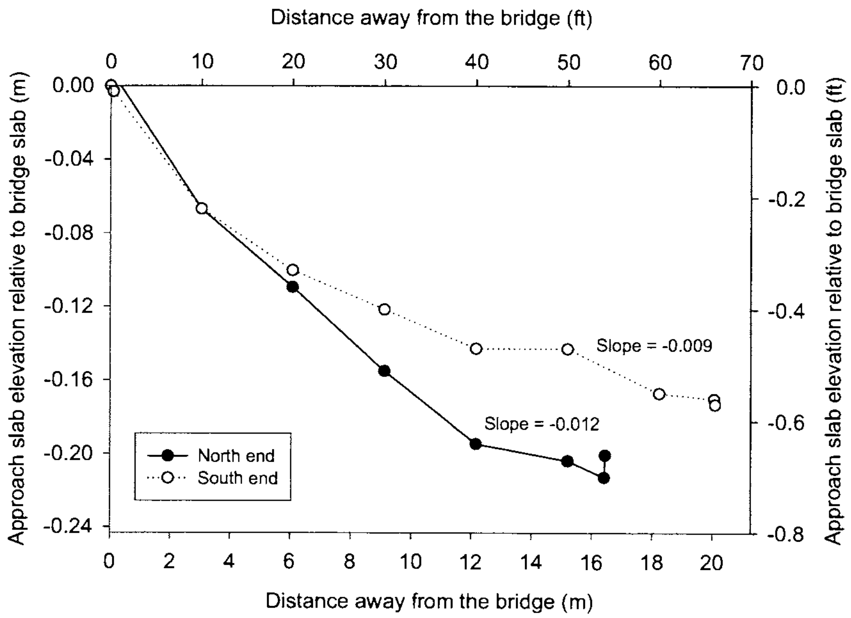


Figure 3.268 - Elevation of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7783.1065 - NBL)

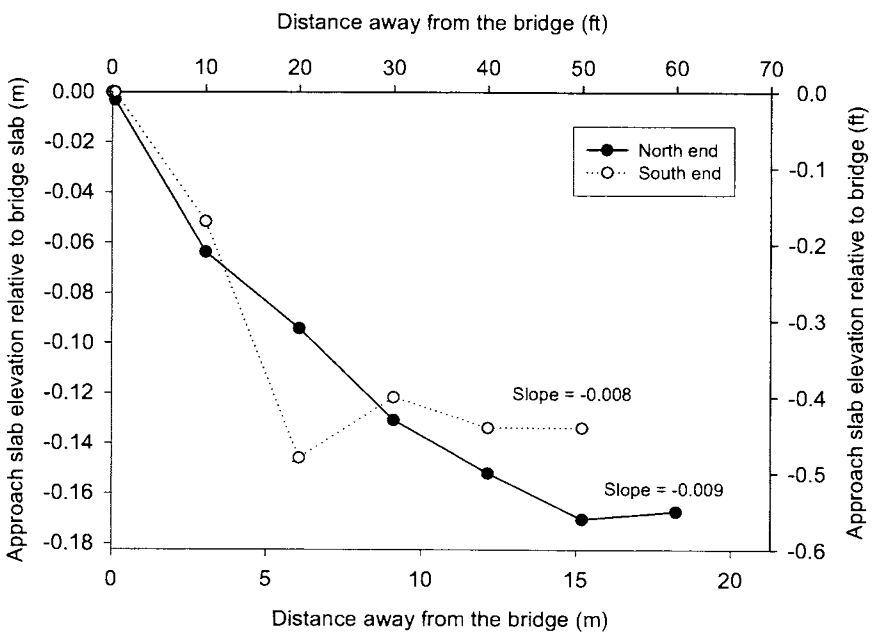


Figure 3.269 - Elevation of the Bridge Approach Relative to the Bridge Deck (Bridge No. 7783.1065 - SBL)

3.2.4 Discussion

From the investigations conducted at existing bridges, it is obvious that water management around bridges in Iowa is a major problem that needs a solution. Many bridges did not have a surface drain. The current DOT surface drain detail shown in Figure 3.49 is not effective in draining the runoff water. However, the surface drain observed at bridge number 5596.2S169 in district 2 (See Figure 3.50) is more effective and helped in reducing erosion around the bridge. Most subdrains behind the abutment were either blocked with soil particles or dry indicating no water flowing through. As a result of poor water management, soil erosion was observed at most inspected bridges. This erosion resulted in increasing the void under the approach slab, faulting of approach slab panels, failure of bridge embankment slope protection due to loss of support, and exposure of H-piles supporting the abutment. Furthermore, the void developed under the approach slab was observed within one year of bridge construction indicating insufficient compaction and poor backfill material. In addition, the flexible foam and recycled tires materials used as joint fillers are not effective in sealing the expansion joints allowing water to flow around the bridge. Grouting behind integral abutment bridges, which was a common practice in district 2 and 6, restrained the abutment movement and caused cracking and spalling of the bridge deck. Moreover, grouting did not prevent either further settlement or loss of backfill material due to erosion.

Monitoring of bridges under construction showed that at most bridges the granular backfill material was placed with poor compaction. Laboratory tests performed on granular backfill, classified as SP, revealed that the material had high compactibility. However, since the Iowa DOT does not specify a moisture content range for placing granular backfill material behind abutments, and the measured field moisture contents were within the bulking moisture content range (i.e., $\approx 3\% - 7\%$), compaction to the specified density was not achieved. In addition, porous backfill was not used around subdrains at most inspected bridges.

Monitoring the maintenance practices in Iowa showed that replacement of bridge approaches occur when the approach slab is severely damaged such as in the case of faulting. Furthermore, the URETEK Inc. method appears to be effective in lifting the approach slab to its original position; however, further monitoring of the approach slabs profiles maintained by URETEK Inc. is needed to observe if additional settlement and erosion are prevented. The U.S. 65 maintenance project revealed that old backfill material under the approach slab was loose and not compacted, and new backfill material was placed without sufficient compaction. The approach slab was observed resting on a very short distance of the old paving notch making the approach slab at risk of sliding off the paving notch.

3.3 Characteristics of Backfill Material

3.3.1 Comparing Backfill Grain Size Distributions to Average Opening of Drainage Pipe

Figure 3.270 shows the perforated pipe used for drainage around bridge abutments in Iowa. The widths of ten random pipe openings were measured and the average and the largest openings of the pipe were determined. The largest pipe opening was 2.32 mm (0.09 in.), and the average pipe opening was 2.01 mm (0.07 in.) which are almost the same size of the No. 8 sieve (2.36 mm). However, Iowa DOT specifies that the percentage passing the No. 8 sieve for granular backfill materials can range from 20% to 100%. Table 3.9 compares the percentage of granular backfill that is finer than the average pipe opening for inspected bridges. The average percent of granular backfill material that could pass through the pipe openings is 72%.

(a) Perforated Drainage Pipe

(b) Closer View of Pipe Openings

Figure 3.270 - Perforated Pipe used in Drainage

Table 3.9 - Comparing Backfill Grain Sizes to the Average Pipe Opening

Bridge location	**Backfill type**	**Moisture content percent**	**Classification**	**Percentage finer than the average pipe opening**
35^{th} St.	Porous	4.15	GP	1
35^{th} St.	Granular	3.9	SP	81
Polk Blvd.	Granular	4.6	SP	78
19^{th} St.	Granular	5.0	SP	79
East 12^{th} St.	Granular	-	SP	78
Bridge over Union Pacific RR	Granular	4.1	SP	84
Bridge no. 57.6R030	Granular	-	SP	30

3.3.2 Collapse Index Test for Granular Soils

Iowa DOT does not specify a moisture content range for the backfill materials used at the bridge abutments. During the field visits to under construction bridges, it was observed that the backfill material was placed at a moisture content ranging from 3.9% to 5.0% (See Table 3.9), which is within the bulking moisture content range (i.e., 3% to 6%) for granular backfill. Furthermore, the backfill material was placed without compaction at most visited under construction bridges. Granular materials undergo settlement (i.e. collapse) when saturated which is governed by the moisture content at which the backfill material was placed and the compaction energy. Therefore, the collapse index test was developed and performed to evaluate the collapse that the backfill material experience when saturated. The test is repeated at initial different moisture contents, and the total change in height at each moisture contents was recorded.

Experiment

Figure 3.271 shows the assembled apparatus to measure the collapse index for various backfill materials. The apparatus consists of Plexiglas cylinder which is open from both ends with a diameter of 20 cm, a height of approximately 94.8 cm, and a 25 mm (1 in.) sieve mounted at 36.6 cm above the cylinder.

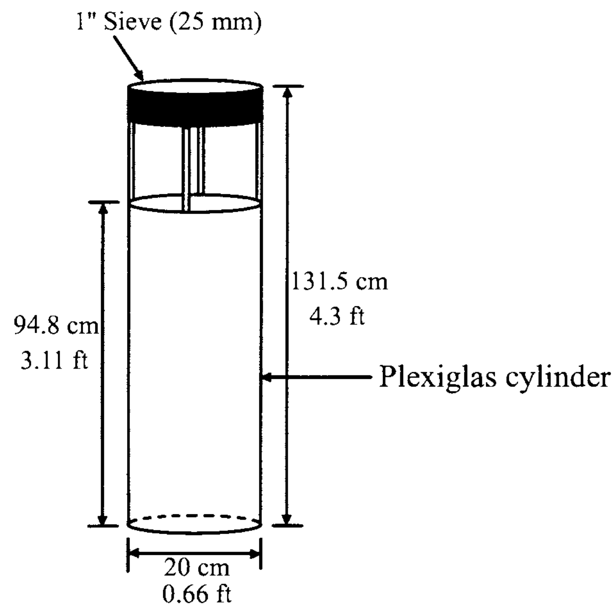


Figure 3.271 - Assembled Apparatus to Measure Collapse Index

Test procedures:

1. The backfill material being tested is dropped at.a given moisture content through the 25 mm sieve at a drop height of 131.5 cm.
2. Gently strike the 4 sides of the cylinder. Refill the cylinder with additional fill material if necessary.
3. Record the height of the backfill material (L).
4. Saturate the fill material by adding water from the top of the cylinder.
5. Keep adding water unit the material is saturated and water flows out of the bottom of the cylinder.
6. Record the backfill material drop in height.
7. Add more water and record any additional drop. Record the total change in height (ΔL).
8. Calculate the collapse index using equation (3.1)

$$\text{Collapse index} = \left(\frac{\Delta L}{L}\right) \times 100 \qquad (3.1)$$

Results

Granular Backfill

The tested material was granular backfill obtained from Hallett Materials Quarry which is similar to granular backfill material observed at many bridge sites. Figure 3.272 compares the gradations of tested the granular backfill material and the granular backfill materials used at four visited under construction bridges. This figure shows that the tested material has similar gradation to the material used at these bridges. The tested granular backfill is classified as poorly graded sand (SP) according to the Unified Soil Classification System. As shown in Figure 3.273, the test was performed at various initial moisture contents and the highest collapse index was 6% achieved at an initial moisture content of 6%.

Porous Backfill

The tested material was classified as poorly graded gravel (GP) according to the Unified Soil Classification System (See Figure 3.274). The material did not settle at different moisture contents. As a result, the material is not expected to settle due to saturation when used as a backfill material behind the abutment; however, the material has low compactibility (See section 3.2.2).

3.3.3 Range of Erodible Soils

Briaud *et al.* (1997) provided a range of soils that are more susceptible to erosion (See Figure 3.275). Soils with silt and fine sand are more erodible that other soil types. This erodible soil range was compared with the Iowa DOT granular backfill gradation requirement (See Figure 3.276) and backfill materials collected at four bridge sites (See Figure 3.277). Both the gradation specified by Iowa DOT and gradation of backfill materials collected at under construction bridges have a common region with the range of most erodible soils. However, changing Iowa DOT specification of granular backfill material from 20%-100% passing the No. 8 sieve (2.36 mm) to 20%-60% will shift the backfill material gradation out of the most erodible soil region.

On the other hand the Iowa DOT gradation requirement for porous fill and the porous fill sample obtained from bridge on 35th St. over I-235 were out of the most erodible soils range (See Figure 3.278). The porous fill therefore could be used as a substitute for granular backfill since it would not settle when saturated nor erode when subjected to runoff water around the abutment.

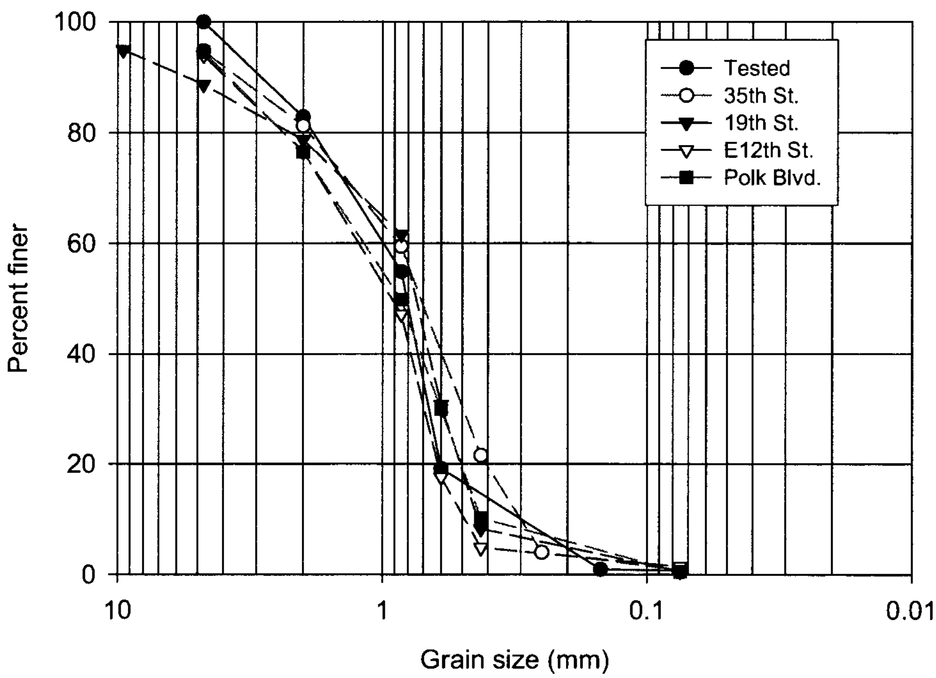


Figure 3.272 - Gradation of Tested Granular Backfill Materials Compared with the Gradation of Samples Collected at Four Under Construction Bridges

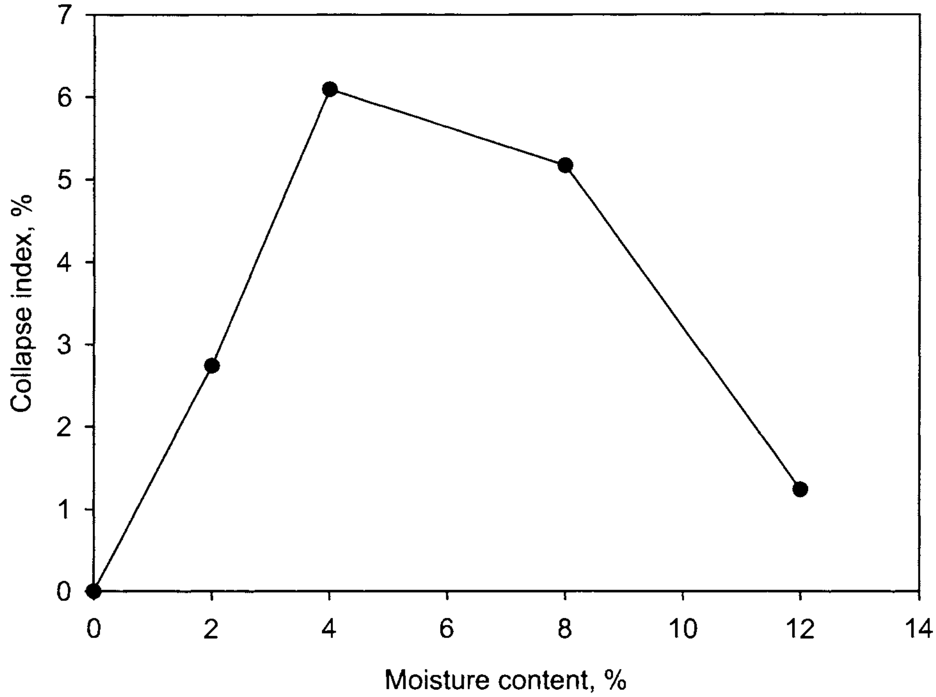


Figure 3.273 - Collapse Index – Moisture Content Relationship for Granular Backfill Material

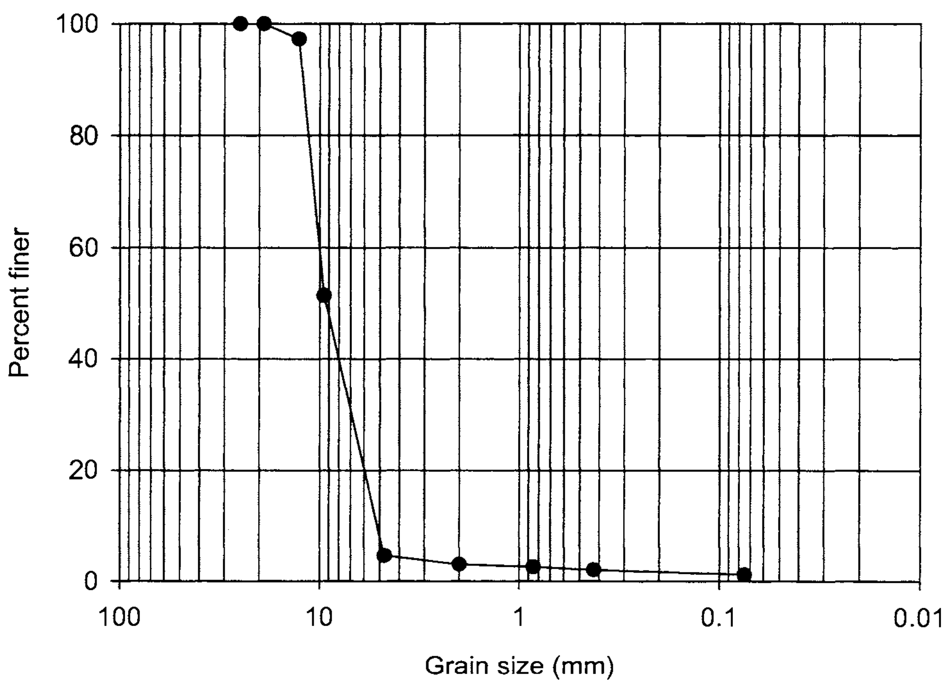


Figure 3. 274 - Grain Size Distribution for Porous Backfill

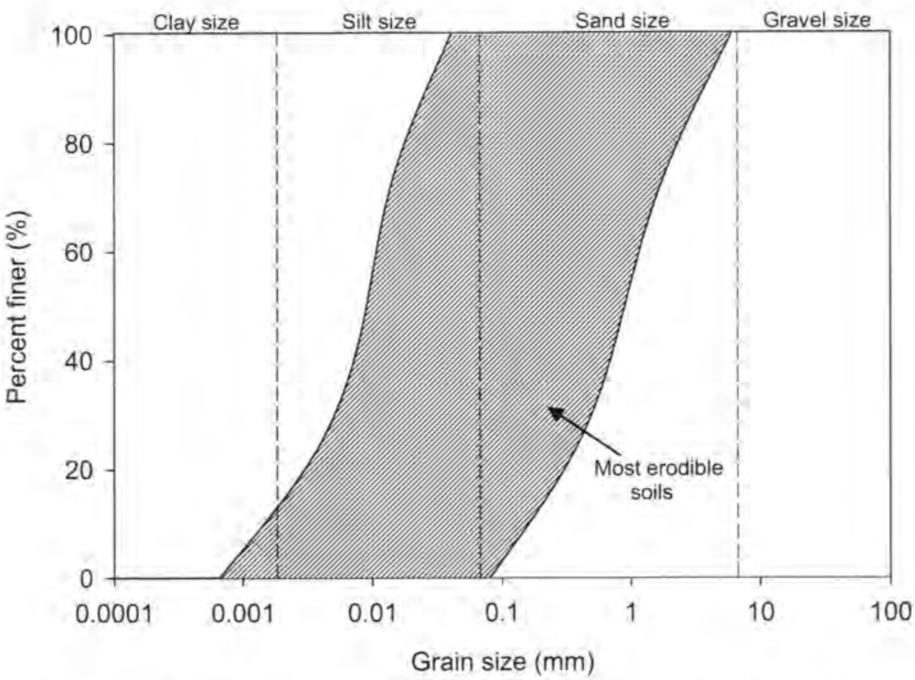


Figure 3.275 - Range of Most Erodible Soils (Briaud *et al.* 1997)

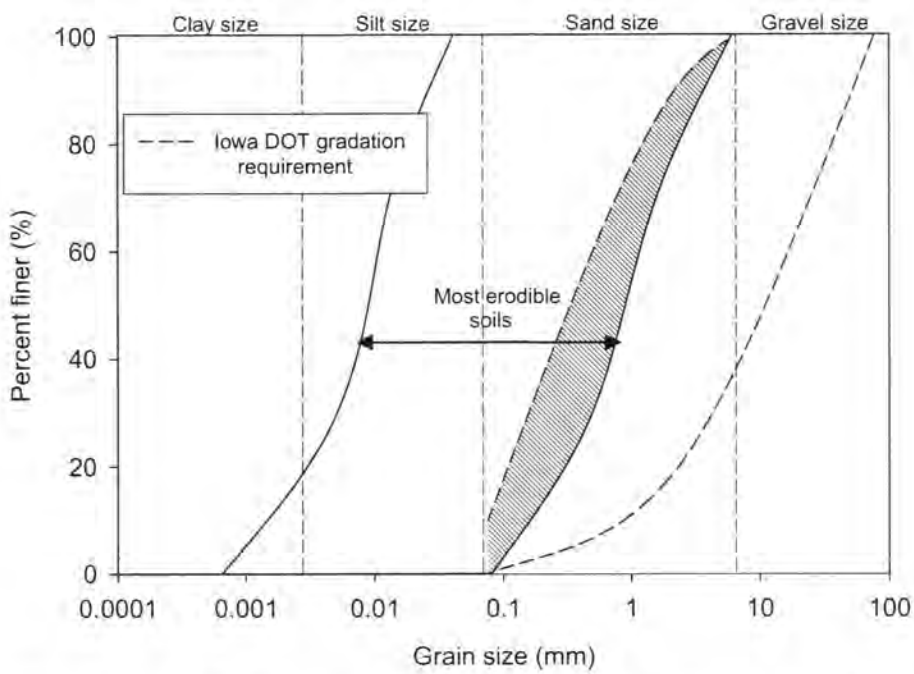


Figure 3.276 - Iowa DOT Granular Backfill Gradation Requirement Compared to the Range of Most Erodible Soils

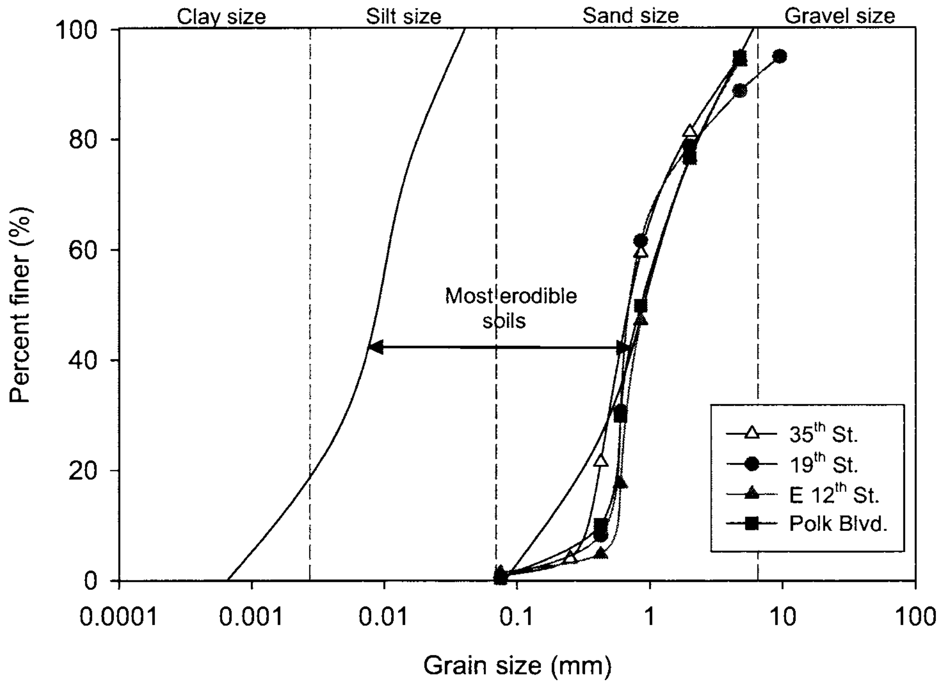


Figure 3.277 - Granular Backfill Obtained from Bridge Sites Compared to the Range of Most Erodible Soils

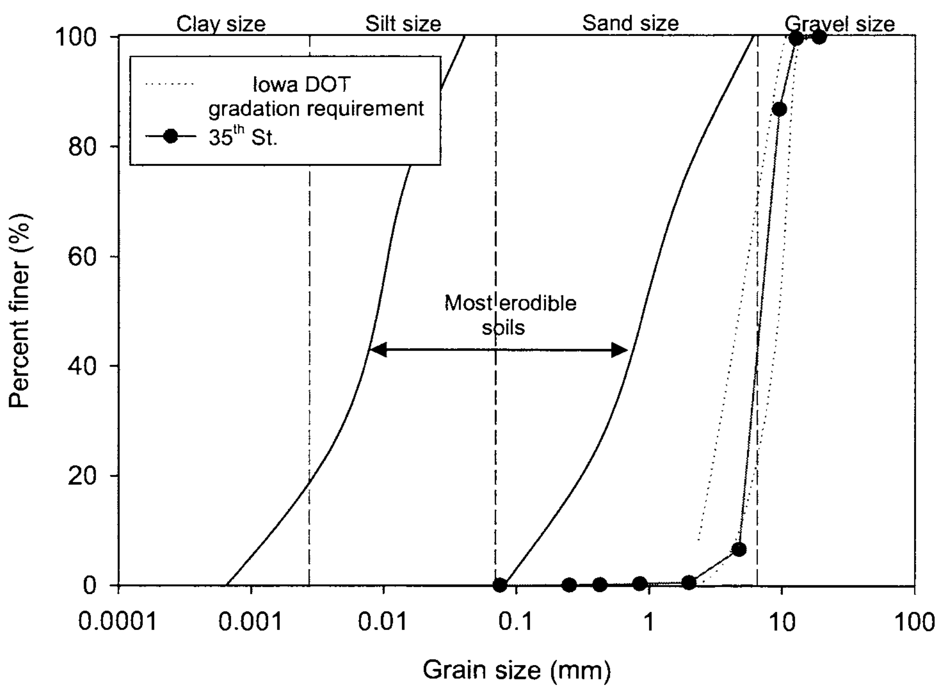


Figure 3.278 - Porous Fill (Pea Gravel) Compared to the Range of Most Erodible Soils

3.3.4 Discussion

After comparing the grain size distribution of the backfill materials to the average opening of the subdrain pipe, it is found that on average 72% of the granular backfill material used behind bridge abutments, and approximately 1% of the porous backfill used around subdrains are smaller than the drainage pipe opening.

The collapse index test showed that the granular backfill material when placed at moisture contents within the bulking moisture content range (3% to 7%) undergoes 6% collapse (settlement) upon saturation. However, granular backfill placed at higher moisture contents (greater than 8%) experience no collapse. Unlike granular backfill, porous backfill material at different moisture contents (0% to 12%) does not experience any collapse when saturated.

The gradation range of granular backfill material specified by Iowa DOT falls within the range of most erodible soils. The gradation of the porous backfill material does not fall within the range of most erodible soils. Therefore, the porous backfill is a good candidate to substitute for granular backfill behind bridge abutments because it is neither a collapsible nor an erodible soil.

3.4 Water Management Bridge Approach Model

Poor water management on and around the bridge is a major problem observed at many bridge sites. It is believed that inadequate drainage is one of the primary problems that needs to be addressed to improve the performance of approach slabs. Poor water management can lead to problems such as void development under the approach slab due to backfill material collapse and erosion which can result in faulting of approach slab panels. Poor drainage can also lead to erosion of bridge embankment material which can result in failure of the slopeprotection and exposure of piles. Therefore, the water management bridge approach model was developed to address this problem. This one-fourth scale model focused primarily on the efficiency of various drainage designs as well as the backfill characteristics. Movement of the bridge structure due to thermal gradient was not tested.

3.4.1 Objectives

The main objectives for constructing the water management bridge approach model were to demonstrate:

- The inadequacy of the current drainage and backfill field practice.
- The performance of the current Iowa DOT drainage and backfill specifications.
- The impact of using various backfill and drainage alternatives based on previous related research, and practices of other states.

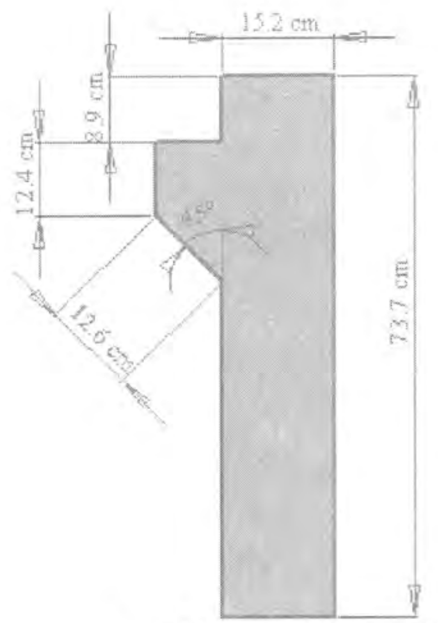
3.4.2 Description of Model

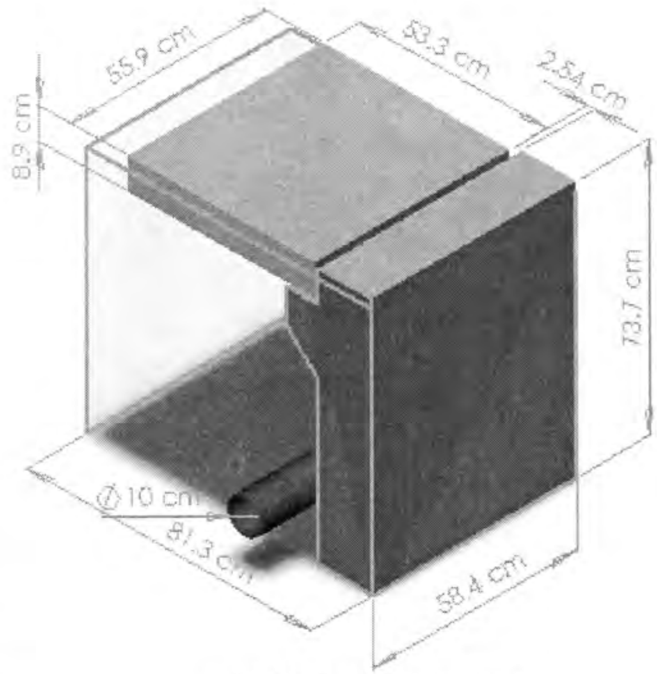
The Water Management Bridge Approach Model consists of an approach slab, abutment, and a drainage pipe. The model was scaled to 25% of the original dimensions except for the drainage pipe which was full scale (See Figures 3.279 and 3.280). Plexiglass was placed at the sides and the bottom of the abutment to retain the backfill material. A perforated HDPE pipe, similar to the ones used in the field, with a 10.2 cm diameter was used as a subdrain. The joint width between the approach slab and the abutment was 2.54 cm to resemble the 10.2 cm expansion joint specified by Iowa DOT. The center of the drainage pipe was positioned at 12.7 cm (5 in.) away from the abutment and 7.6 cm (3 in.) above the bottom of the model. The model was 73.7 cm (29 in.) high, 58.4 cm (23 in.) wide and 81.3 cm (32 in.) long.

Water flowed through the expansion joint, under the approach slab, through the drainage system, and out of the subdrain. Water was collected in a trench around the model then pumped back into the model using a submerged pump. To disperse the water coming into the model, a perforated tank was placed on top of the expansion joint.

To compare different drainage details, each test was allowed to run for the same time period (i.e., four hours). Settlement, size of void development, and maximum steady state water flow rate were determined. Settlement was calculated by measuring the difference between the approach slab elevation before and after the test. Void dimensions under the approach slab were recorded. Furthermore, the time needed for water to come out of the subdrain was recorded, and the maximum steady state flow for each design was calculated. The inlet flow was altered as necessary until a steady state condition was reached. Once a steady state flow is reached, the flow is fixed at this steady condition until the end of the test and the flow was recorded.

Due to the scale of the model and the difference in boundary conditions between the model and the bridge approach slabs, the model focused primarily on comparing drainage details. The model was not used to predict behavior of approach slabs in the field.





(a) Abutment Dimensions **(b) Model Dimensions**

Figure 3.279 - Schematic for the Constructed Water Management Bridge Approach Model

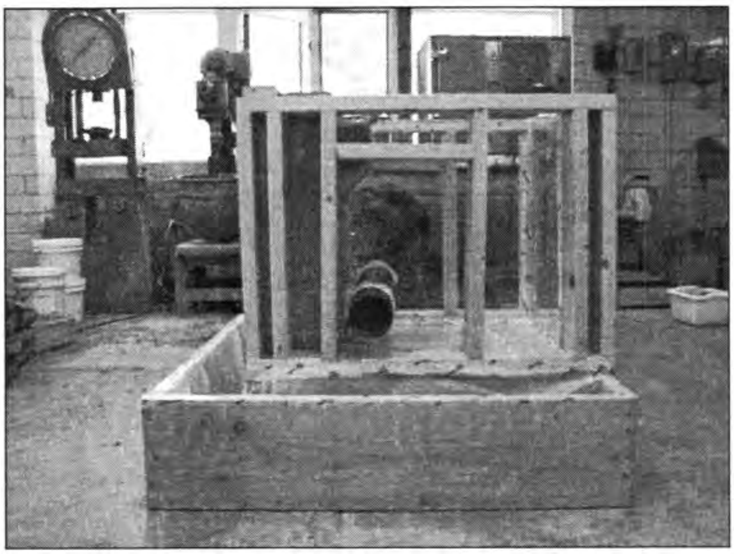


Figure 3.280 - Water Management Bridge Approach Model

3.4.3 Backfill Materials

Figures 3.281 through 3.284 presents the grain size distributions of the granular backfill, porous backfill (Pea gravel), special backfill, and tire chips used throughout the model testing. The granular and the porous backfill materials were the same as the materials used in the collapse index testing. The granular backfill was classified as SP with a bulking moisture content range of 3% to 6%. The porous backfill and the special backfill were classified as GP. Granular, porous, and special backfill materials meet the Iowa DOT gradation requirement (Refer to section 4109.02 in Iowa DOT Standard Specifications for Highway and Bridge Construction).

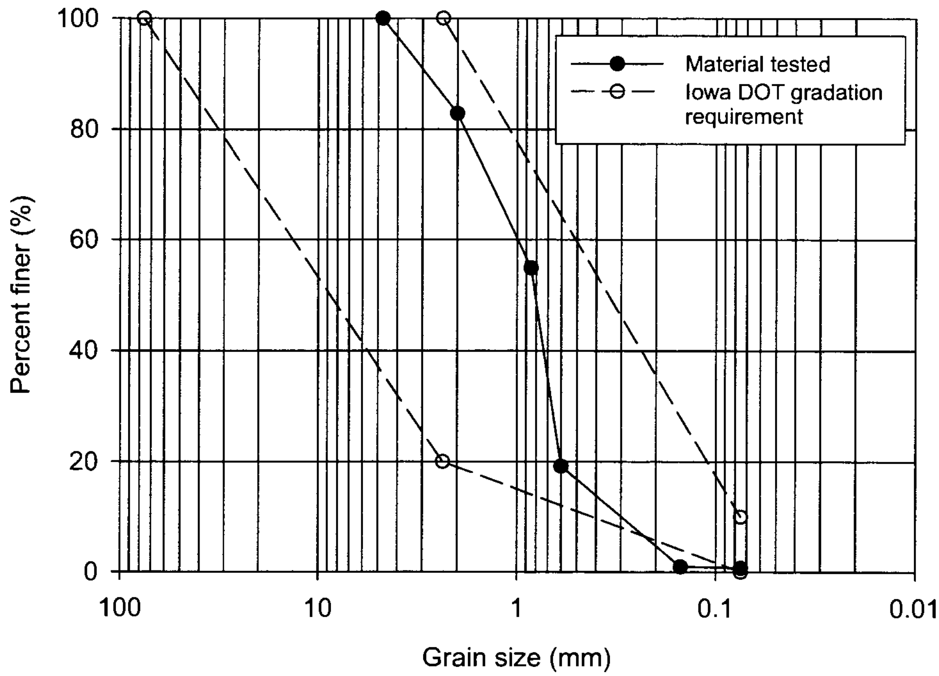


Figure 3.281 - Grain Size Distribution for Granular Backfill

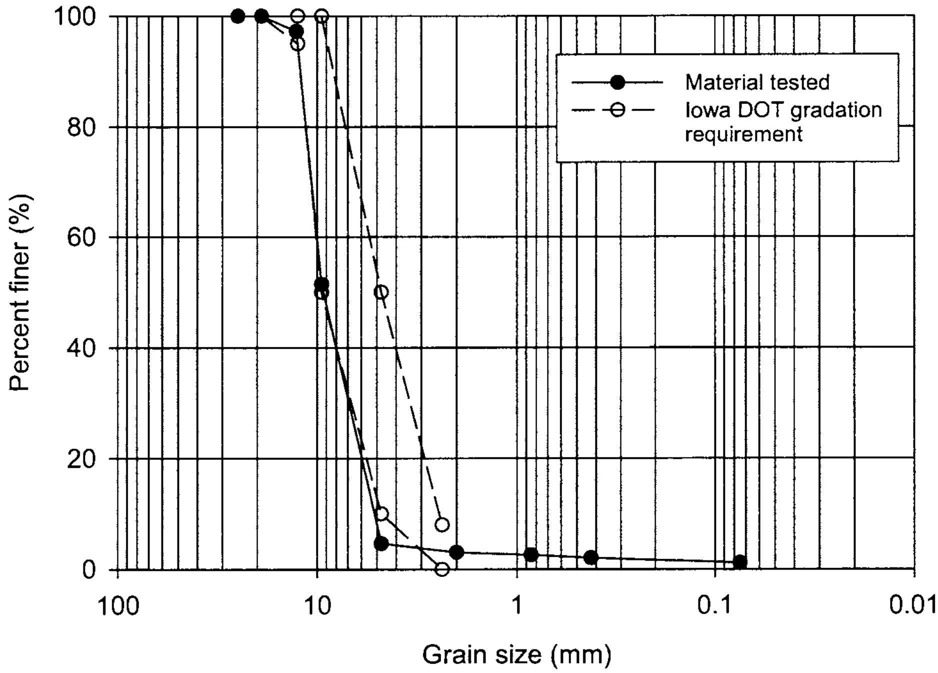


Figure 3.282 - Grain Size Distribution for Porous Fill

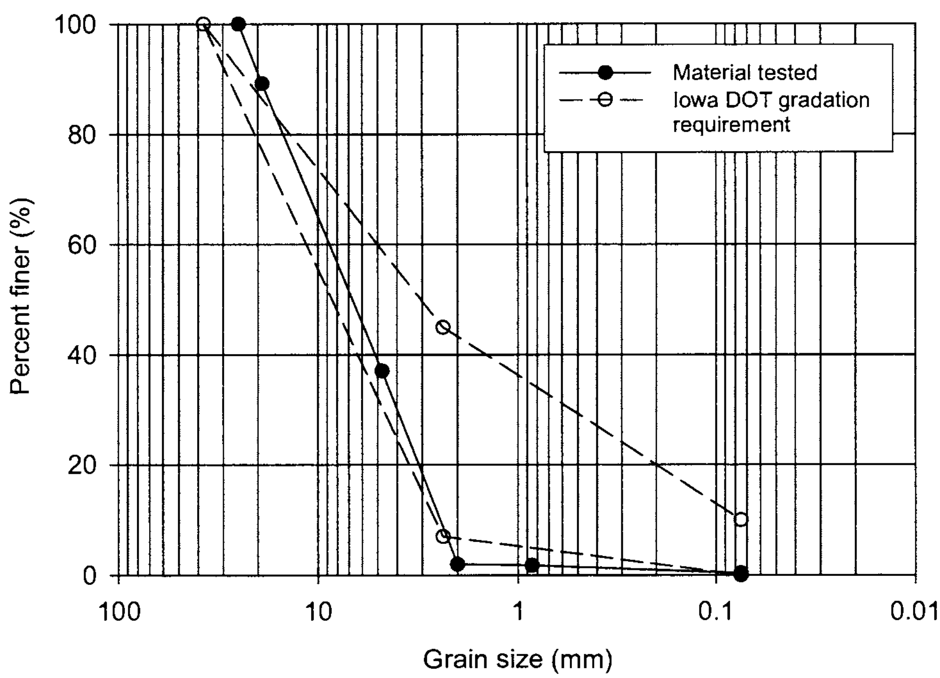


Figure 3.283 - Grain Size Distribution for Special Backfill

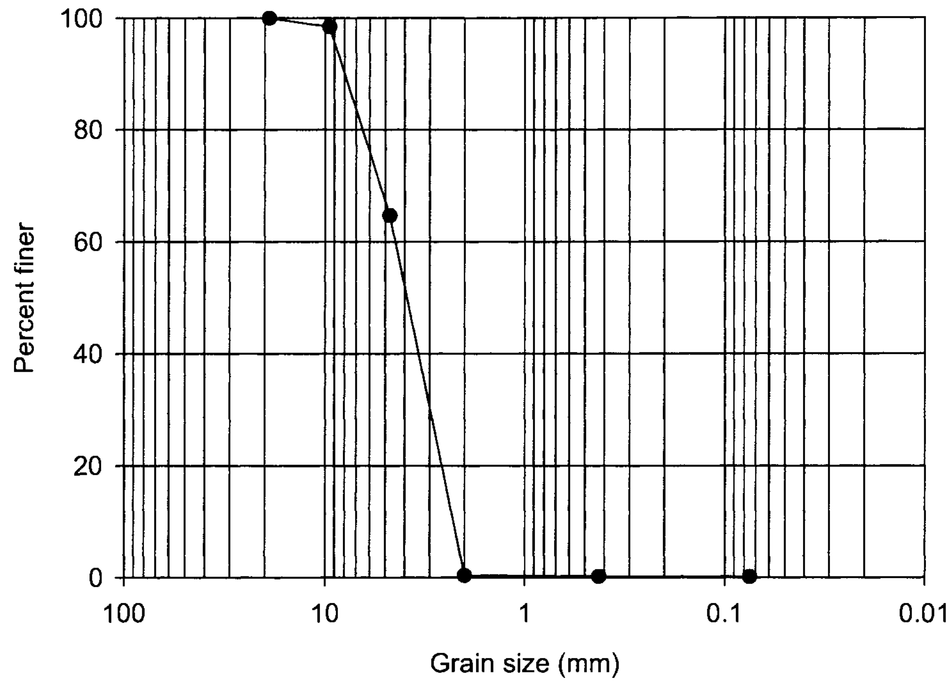


Figure 3.284 - Grain Size Distribution for Tire Chips

3.4.4 Test 1: Current Iowa DOT Drainage Detail (3.0% Moisture Content)

The purpose of this test was to evaluate the current Iowa DOT design. It is specified in Iowa DOT Bridge Standards Sheet no. 2078 that the porous backfill shall cover the subdrain by a minimum of 100 mm and extend to 660 mm from the abutment. For the model, the porous fill was placed to a height of 2.5 cm above the subdrain and the thickness of the porous fill layer was 16.5 cm (See Figure 3.285). Furthermore, granular backfill was compacted every 50 mm lift with a tamper simulating compaction every 200 mm specified in Iowa DOT Standard Specifications for Highway and Bridge Construction section 2107. A 7.6 cm layer of special backfill (crushed limestone) was placed above the granular backfill with a geogrid placed between the special and granular backfill (See Figures 3.285 and 3.286). The geogrid used was a structural geogrid BX1100. (See Appendix A for properties of the geogrid as specified by the manufacturer). Iowa DOT does not specify moisture content for granular backfill; the backfill was therefore placed at bulking moisture consistent with the observations made during field visits to under construction bridges (≈ 3%).

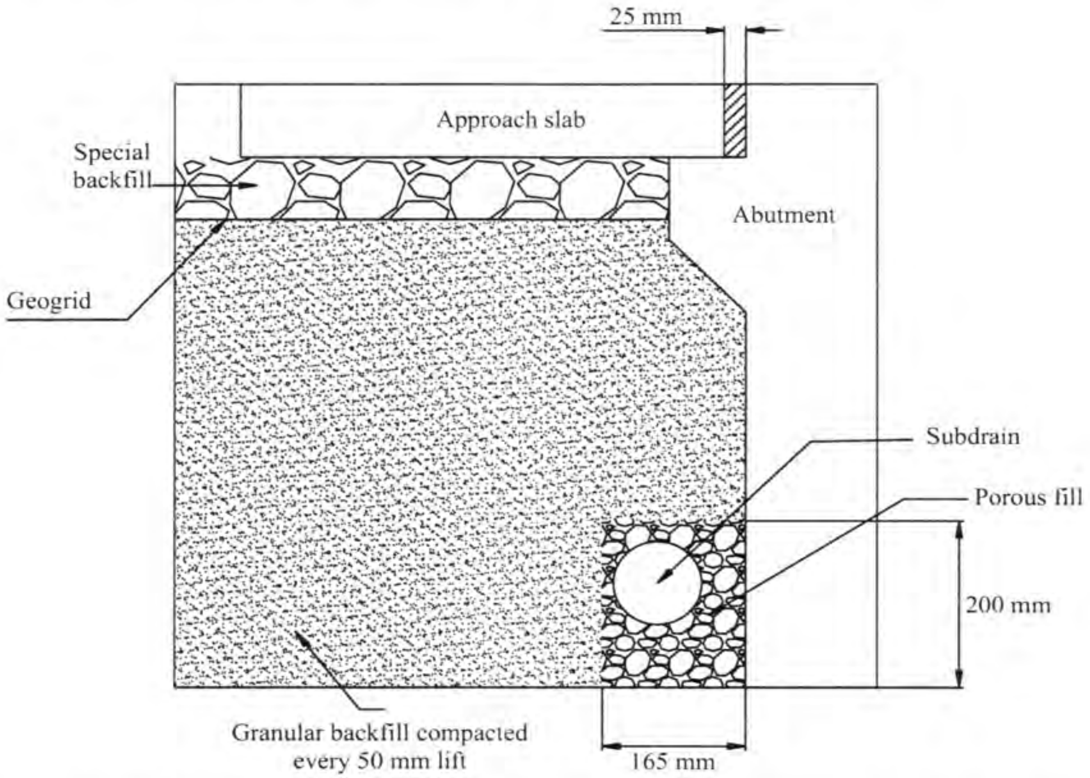


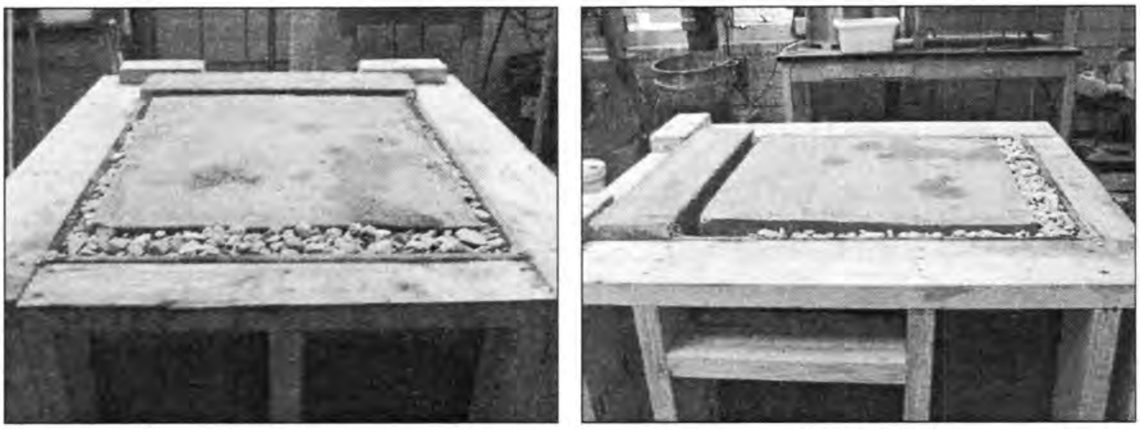
Figure 3.285 - Schematic Diagram of Test 1 Representing Iowa DOT Current Design

Figure 3.286 - Geogrid Placed under Special Backfill

Water started to flow out of the subdrain after 10 minutes from starting the test with fines observed being washed out. After 15 minutes from the beginning of the test, no more fines were washed out and the drained water was clear. Table 3.10 summarizes the results of this test. The maximum achieved steady state flow was 32 cm^3/sec. The maximum void developed was 11.4 cm and extended the full width of the approach slab and was largest at the abutment face, while the maximum differential settlement was 5.1 cm (See Figures 3.287 and 3.288). Drainage occurred from the bottom as water rose and filled the subdrain. A possible explanation is that the medium to coarse sand particles plugged the pore spaces between the porous fill at the upper portion of the subdrain.

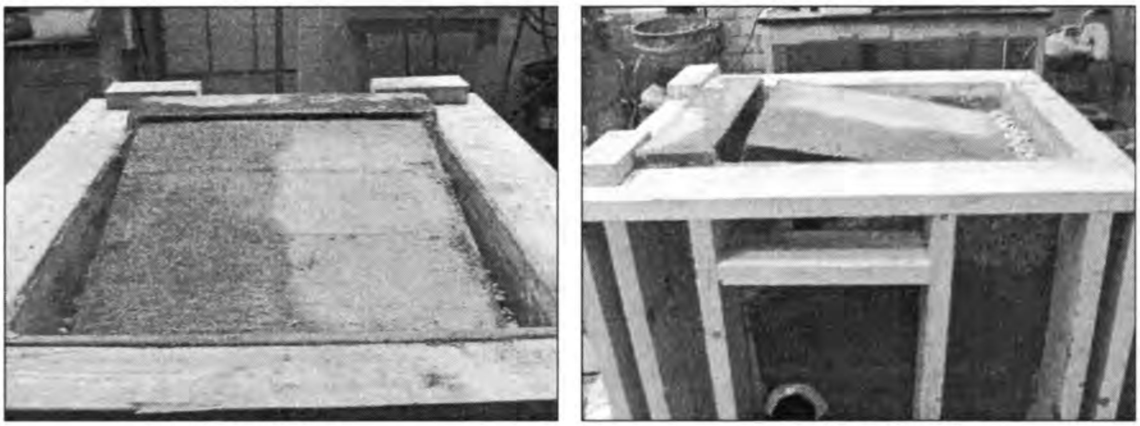
Table 3.10 - Key Results from Test 1

Backfill type	Granular backfill with porous fill around subdrain and special backfill under the approach slab
Moisture content	3.0 %
Compaction	By tamper every 50 mm lift
Settlement (cm)	Left side: 5.1 cm
	Right side: 3.8 cm
Void (cm)	Left side: 11.4 cm deep at the abutment face and extending 21.6 cm under the approach slab
	Right side: 8.9 cm deep at the abutment face and extending 22.9 cm under the approach slab
Maximum steady state flow (cm^3/sec)	32.0
Time for water to drain (min)	10



(a) Front View (b) Side View

Figure 3.287 - Position of Bridge Approach before Test 1



(a) Front View (b) Side View

Figure 3.288 - Position of Approach after Test 1

3.4.5 Test 2: Current Iowa DOT Drainage Detail with Saturated Granular Backfill

The purpose of this test was to observe the effect of altering the granular backfill moisture content on the approach slab settlement, void development, and maximum steady flow. The granular backfill was placed at 12.6% moisture, and compacted every 50 mm lift using a tamper. This moisture content was based on the collapse index test performed on granular backfill, which showed no collapse at moisture content between 8 and 12% (See section 3.3.2).The current Iowa DOT design detail was used as shown in Figure 3.289 (See setup of test 1).

Figure 3.289 - Placing Special Backfill over Geogrid

Water started to drain after 12 minutes from starting the test, and fines were observed being washed out. After 17 minutes, no more fines were washed out and the water drained was clear. Table 3.11 summarizes the results of this test. Saturation and compaction of the granular backfill prevented the bridge approach settlement and the void formation after 4 hours of testing (See Figures 3.290 and 3.291). However, the maximum achieved steady state flow was 31 cm^3/sec. Drainage occurred as water rose from the bottom and filled the subdrain, which indicate that medium to coarse sand particles plugged the drainage pipe openings or soil compression above the subdrain prevented drainage from the top portion of the drain. This is expected to occur only in the model since in the field the water will seep through the soil particles and will not rise to the subdrain as observed in the model.

Table 3.11 - Key Results from Test 2

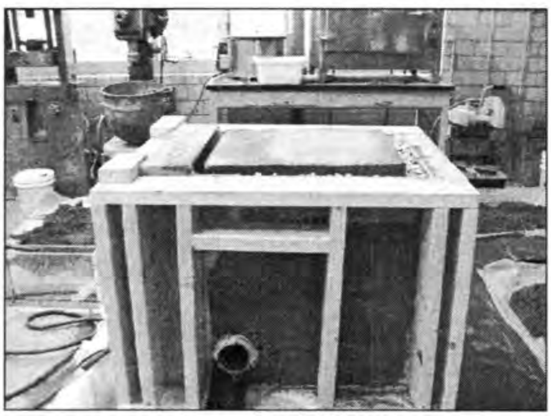
Backfill type	Granular backfill with porous fill around the subdrain and special backfill under the approach slab
Moisture content	12.6 %
Compaction	By tamper every 50 mm lift
Settlement (cm)	No settlement
Void (cm)	None
Maximum steady state flow (cm^3/sec)	31
Time for water to drain (min)	12

(a) Front View

(b) Side View

Figure 3.290 - Position of Approach Slab before Test 2

(a) Front View



(b) Side View

Figure 3.291 - Position of Approach Slab after Test 2

3.4.6 Test 3: Current Field Practice-1

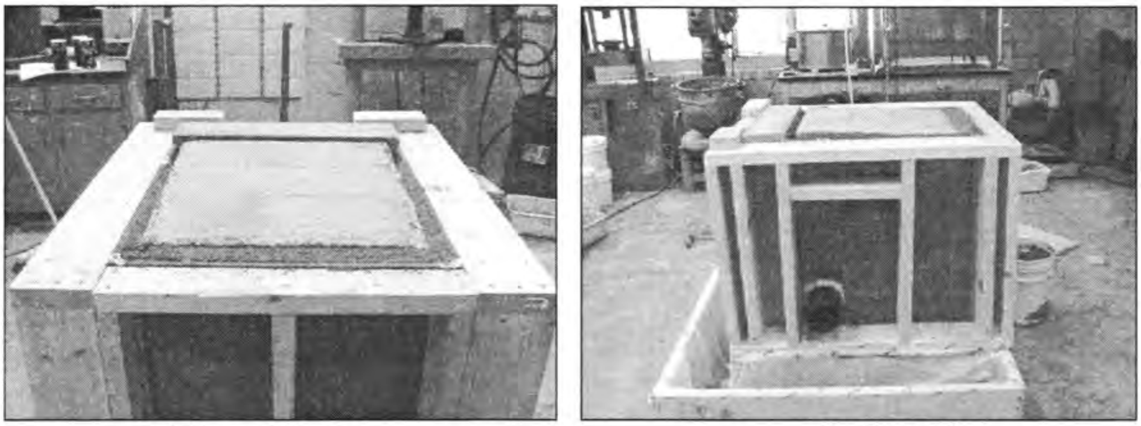
The purpose of this test was to recreate the construction practices observed during field visits for under construction bridges. The backfill was placed at an average moisture content of 3%, which is within the bulking moisture content range, and compacted by its own weight to simulate dumping the backfill behind the abutment with no compaction effort. Furthermore, no porous backfill was placed around the subdrain (See Figure 3.292).

Figure 3.292 - Granular Backfill Placed behind the Abutment

The water started to flow out of the subdrain after 10 minutes from starting the test. Fine sand particles were washed out of the subdrain at the beginning of the test. Water started to clear out indicating less fine sand washed out after 20 minutes from running the test. The void formed extended through the full width of the approach slab. The maximum void depth was 10.2 cm at the abutment and extended 11.4 cm away from the abutment. The highest soil collapse occurred above the subdrain. The maximum settlement measured from the edge of the approach slab was 5.7 cm (See Figures 3.293 and 3.294). Drainage occurred as water rose form the bottom and filled the subdrain, which is a phenomenon associated only with the model. Table 3.12 summarizes the results of this test.

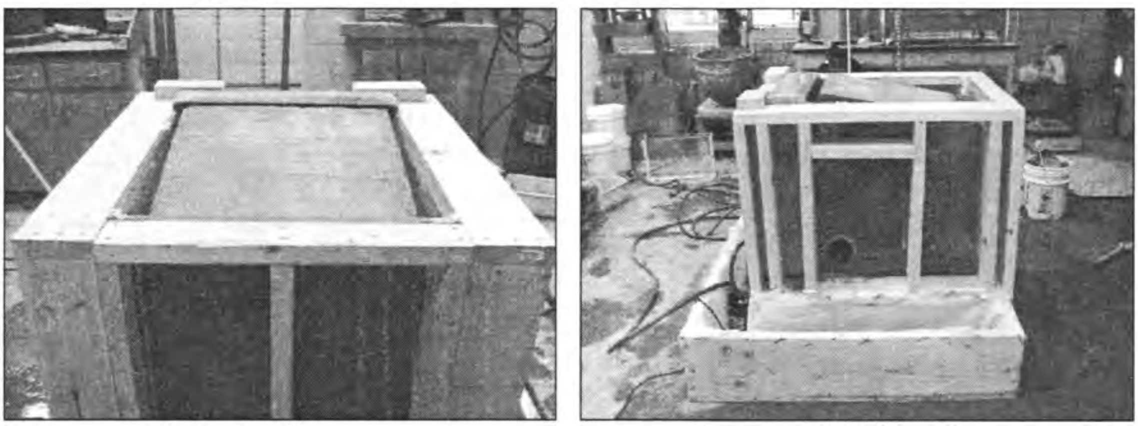
Table 3.12 - Key Results from Test 3

Backfill type	Granular
Moisture content	3.0 %
Compaction	By own weight
Settlement (cm)	Left side: 4.4 cm
	Right side: 5.7 cm
Void (cm)	Left side: 10.2 cm deep at the abutment face and extending 11.4 cm under the approach slab
	Right side: 8.3 cm deep at the abutment face and extending 11.4 cm under the approach slab
Maximum steady state flow (cm^3/sec)	33.5
Time for water to drain (min)	10



(a) Front View **(b) Side View**

Figure 3.293 - Position of Bridge Approach before Test 3



(a) Front View **(b) Side View**

Figure 3.294 - Position of Approach Slab after Test 3

3.4.7 Test 4: Current Field Practice-2

The purpose of this test was to recreate the field practice observed at two under construction bridges where porous backfill was used around the subdrain; however, the granular backfill was not compacted. The porous fill was placed to a height of 2.54 cm above the subdrain and the thickness of the porous fill layer was 165 mm (See Figure 3.295). Granular backfill was placed at a moisture content of 5.5 %, which is within the bulking moisture content range. The backfill material was compacted by its own weight to simulate dumping the backfill behind the abutment.

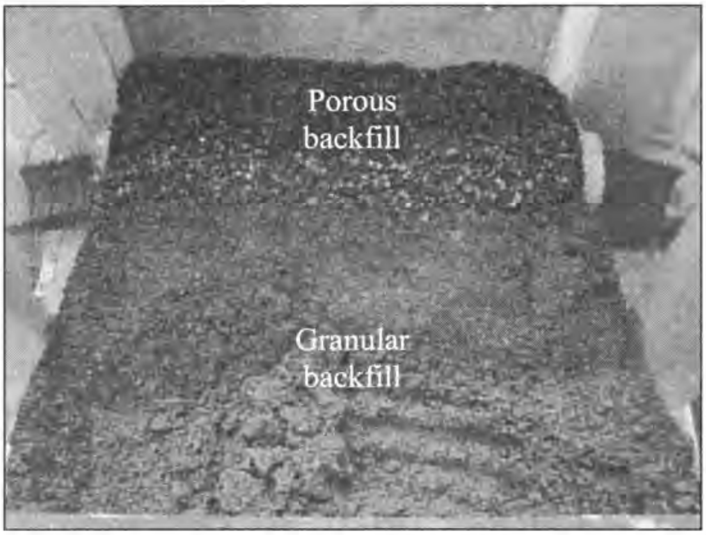


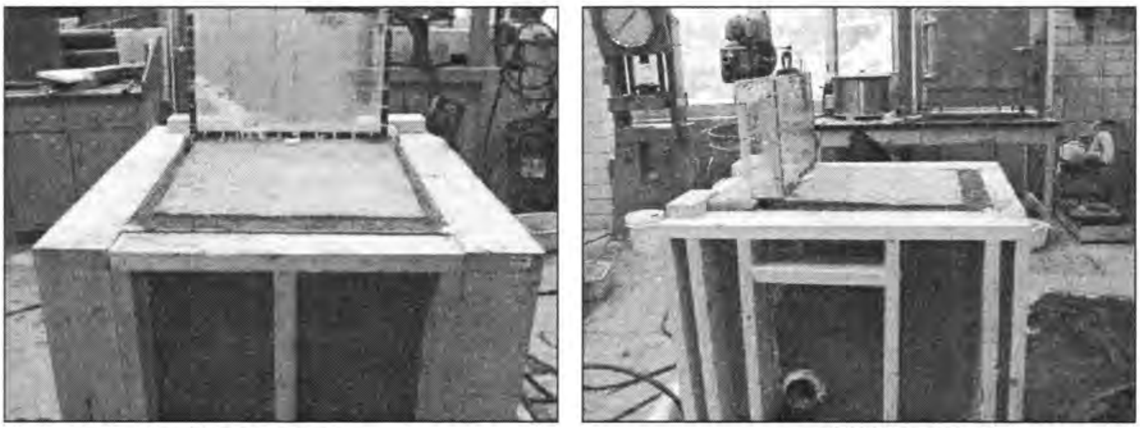
Figure 3.295 - Porous Fill Placed Around the Subdrain

Water started to flow out of the subdrain after 11 minutes from starting the test which is similar to the test 1. However, the maximum steady state flow increased to twice (67 cm^3/sec) the drainage capacity of test. The porous fill decreased the amount of fines that were washed out compared to test 3 where no porous fill was used. Water drained out became clear after 14 minutes from running the test. Drainage out of the subdrain occurred as water rose from the bottom and filled the subdrain. No water was drained from the upper portion of the drain. Table 3.13 summarizes the results of this test. The maximum void developed was

5.1 cm at the abutment that extended 25.4 cm away from the abutment. This void extended through the full width of the approach slab. The maximum soil collapse occurred above the subdrain, and the maximum settlement measured from the edge of the approach slab was 5.7 cm (See Figures 3.296 and 3.297).

Table 3.13 - Key Results from Test 4

Backfill type	Granular with porous fill around subdrain
Moisture content	5.45 %
Compaction	By own weight
Settlement (cm)	Left side: 5.1 cm
	Right side: 5.7 cm
Void (cm)	Left side: 5.1 cm deep at the abutment face and extending 25.4 cm under the approach slab
	Right side: 5.1 cm deep at the abutment face and extending 7.6 cm under the approach slab
Maximum steady state flow (cm^3/sec)	67.0
Time for water to drain (min)	11



(a) Front View **(b) Side View**

Figure 3.296 - Position of Approach Slab before Test 4

(a) Front View

(b) Side View

Figure 3.297 - Position of Approach Slab after Test 4

3.4.8 Test 5: Wrapping the Porous Fill with Geotextile

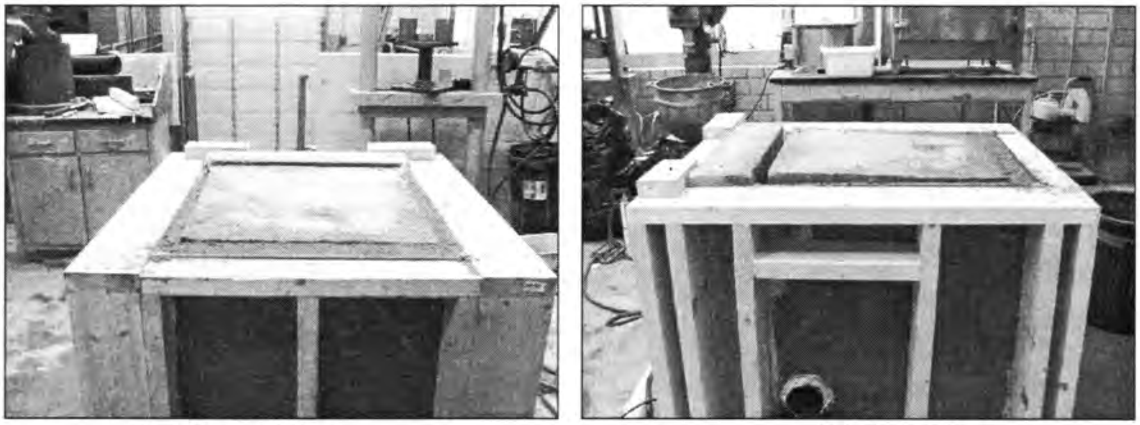
The purpose of this test was to study the effect of adding a geotextile fabric around the porous fill to the amount of fines washed out, settlement, void size, and maximum steady state flow. The setup of this test was similar to test 4 except that the geotextile is wrapped around the porous fill (See Figure 3.298). CONTECH C-60NW nonwoven geotextile was used in this model. CONTECH C-60NW meets the requirements for a class 2 subsurface drainage, separation, and stabilization geotextile according to AASHTO M288-96. (See Appendix A for the CONTECH C-60NW geotextile properties as specified by the manufacturer). The height of the porous fill was 200 mm from the bottom of the abutment and 165 mm wide. Granular backfill was placed at bulking moisture (4.8 %) and compacted by its own weight.

Figure 3.298 - Geotextile Fabric Wrapped Around Porous Fill

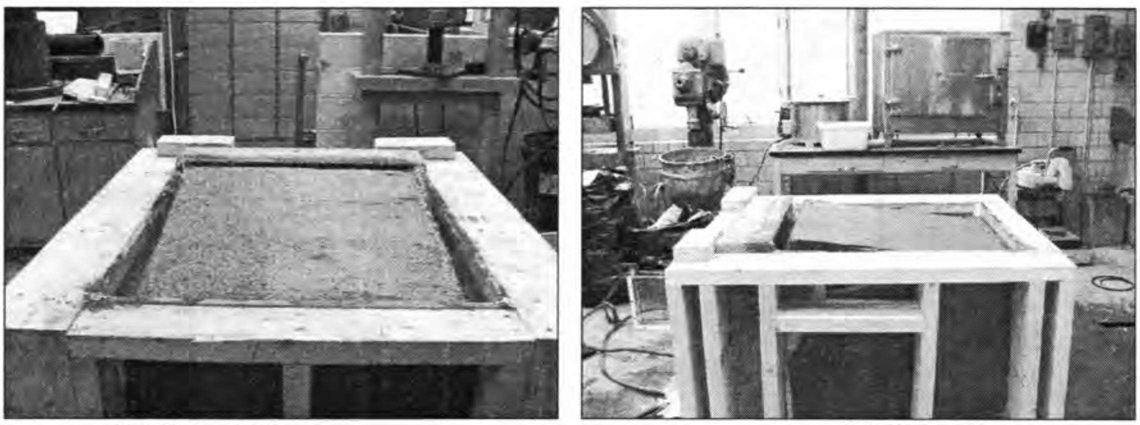
Water started to flow out of the subdrain after 10 minutes from starting the test which is similar to tests 3 and 4. However, the geotextiles increased the maximum achieved steady state flow to 82 cm^3/sec, which represent 22% increase when compared to test 4 (See Table 3.14). In addition, the geotextile decreased the amount of fines washed out since the water cleared after 12 minutes from starting the test. However, some fines were washed out, which is due to the fact that approximately 5% of the granular backfill is smaller than the apparent opening size of the geotextile. The maximum settlement measured from the edge of the approach slab was 5.1 cm (See Figures 3.299 and 3.300). When compared to test 4, using geotextile resulted in a decrease of 16% in void size and 11% in approach slab settlement. The developed void size was 6.4 cm deep that extended 22.9 cm away from the abutment, and through the full width of the approach slab (See Figure 3.301). The maximum soil collapse occurred above the subdrain. Similar to test 3 and 4, drainage occurred from the bottom portion of the subdrain as water rose and filled the subdrain.

Table 3.14 - Key Results from Test 5

Backfill type	Granular with porous fill around subdrain wrapped with geotextiles
Moisture content	4.8 %
Compaction	By own weight
Settlement (cm)	Left side: 5.1 cm
	Right side: 4.4 cm
Void (cm)	Left side: 6.4 cm deep at the abutment face and extending 22.9 cm under the approach slab
	Right side: 4.44 cm deep at the abutment face and extending 21.6 cm under the approach slab
Maximum steady state flow (cm^3/sec)	82.0
Time for water to drain (min)	10



(a) Front View (b) Side View
Figure 3.299 - Position of Approach Slab before Test 5



(a) Front View (b) Side View
Figure 3.300 - Position of Approach Slab after Test 5

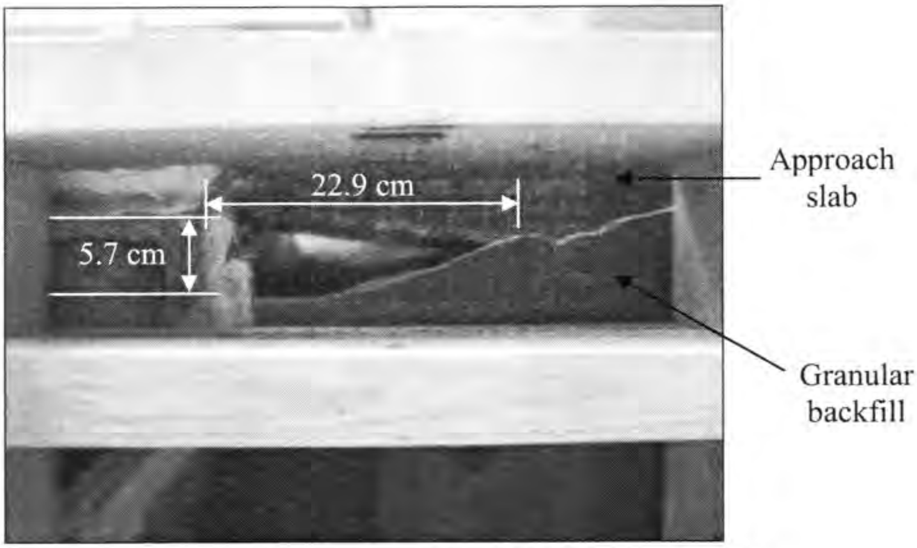


Figure 3.301 - A void Developed under the Approach Slab

3.4.9 Test 6: Geotextile around Porous fill and Backfill Reinforcement

The purpose of this test was to evaluate the effectiveness of backfill reinforcement in alleviating the approach slab settlement and reduction of void size. Figure 3.302 shows a diagram of this test. Geotextile (CONTECH C-60NW) was wrapped around the porous fill as in test 5, and soil reinforcement was placed every 76 mm starting above the porous fill. As shown in Figure 3.303, CONTECH C-80NW was used as soil reinforcements, which meets AASHTO M288-96 requirements for a class 1 permanent erosion control and stabilization geotextile (See Appendix A for detailed properties of CONTECH C-80NW geotextile as specified by the manufacturer). At the abutment face the geotextile was folded around the backfill as shown in Figure 3.302. The length of the embedded geotextile was approximately 130 mm. The porous backfill layer was 200 mm high and 165 mm wide. Granular backfill was placed at bulking moisture (5.2 %) and compacted by its own weight to simulate dumping of backfill behind the abutment.

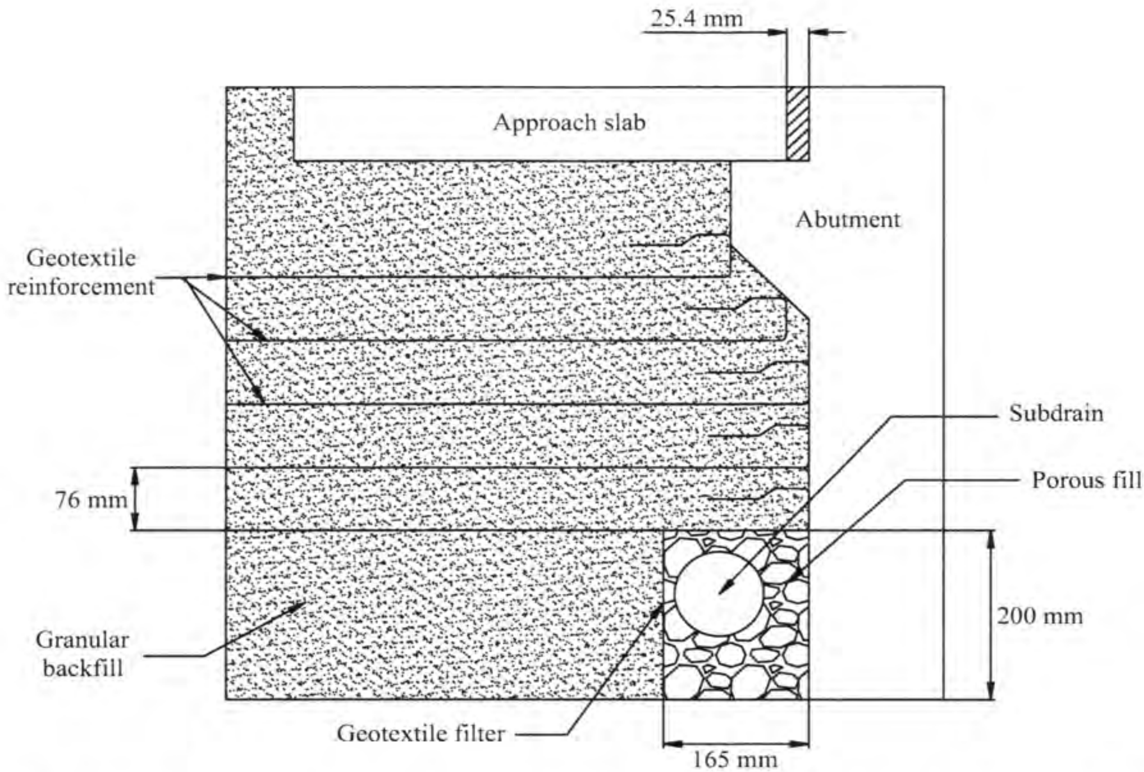


Figure 3.302 - Schematic of Test 6 with Mechanically Stabilized Backfill behind the Abutment and Porous Fill Wrapped with Geotextiles Around the Drainage Pipe

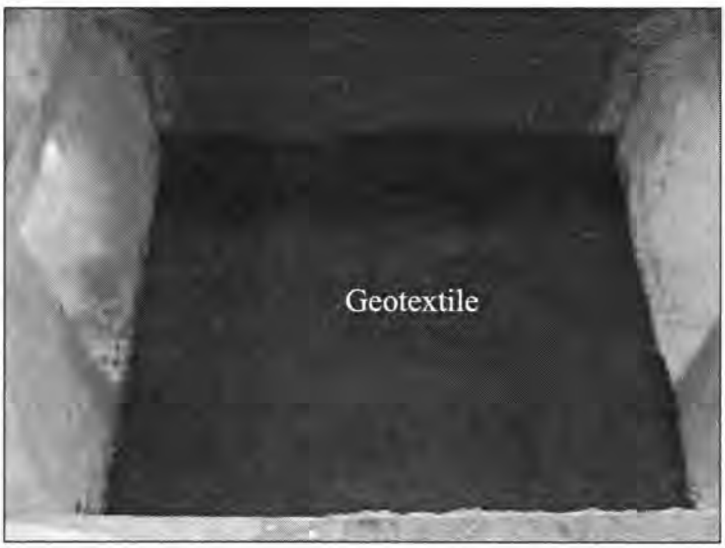


Figure 3.303 - Placing Backfill Reinforcement

Water started to flow out of the subdrain after 7 minutes from starting the test. When compared to test 5, adding backfill reinforcement decreased the void size by 30% and the settlement by 50%. However, the addition of the reinforcement decreased the maximum steady state flow by 23% compared to test 5, but it was still 100% higher than the test 3 which was simulating field practices. Table 3.15 summarizes the results of this test. The developed void was 4.4 cm at the abutment and extended 16.5 cm away from the abutment (See Figure 3.304). The void was largest at the abutment face and extended the full width of the approach slab. The mechanically stabilized backfill and the approach slab settled 2.5 cm when saturated (See Figure 3.305). Fewer fines were washed out of the subdrain compared to test 5. Similar to previous tests soil compression above the subdrain prevented the water from being drained from the top of the subdrain. Drainage occurred from the bottom as water rose and filled the subdrain.

Table 3.15 - Key Results from Test 6

Backfill type	Reinforced granular backfill with porous fill around subdrain wrapped with geotextiles
Moisture content	5.2 %
Compaction	By own weight
Settlement (cm)	Left side: 2.5 cm
	Right side: 2.5 cm
Void (cm)	Left side: 4.4 cm deep at the abutment face and extending 16.5 cm under the approach slab
	Right side: 3.8 cm deep at the abutment face and extending 19.7 cm under the approach slab
Maximum steady state flow (cm^3/sec)	63.0
Time for water to drain (min)	7

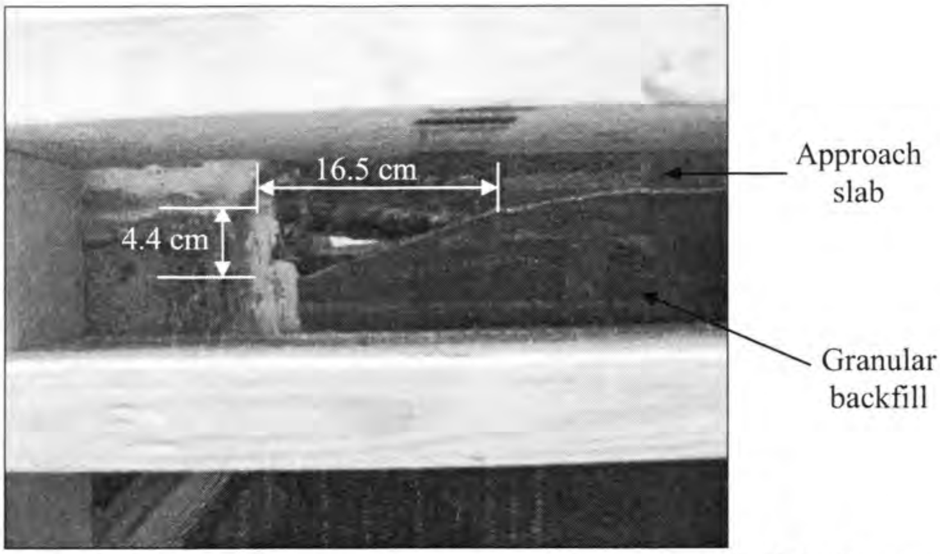


Figure 3.304 - A Void Developed under the Approach Slab Due to Soil Collapse

(a) Front View **(b) Side View**

Figure 3.305 - Position of Approach Slab after Test 6

3.4.10 Test 7: Granular Backfill with Vertical Geocomposite Drainage System and Backfill Reinforcement (Tenax Ultra-Vera™ Geotextile)

The purpose of this test was to evaluate the effect of using a vertical geocomposite drain attached to the face of the abutment on the maximum steady flow, approach slab settlement, and void formation. The vertical drain core used in this test was a Tenax Ultra-Vera™ Geotextile which is still under development by Tenax. The synthetic drain was bolted to the abutment as shown in Figures 3.306. (See Appendix A for properties of the synthetic drain as provided by the supplier). The first backfill reinforcement layer was placed after a 76 mm lift. The following layers were placed every 13 mm lifts (See Figure 3.307). At the abutment face the geotextile was folded and embedded under the backfill. The length of the embedded geotextile was approximately 130 mm. The geotextile used for backfill reinforcement was CONTECH C-80NW. The granular backfill was placed at a bulking moisture content of 4.2% and compacted by its own weight to simulate dumping of backfill material behind the abutment.

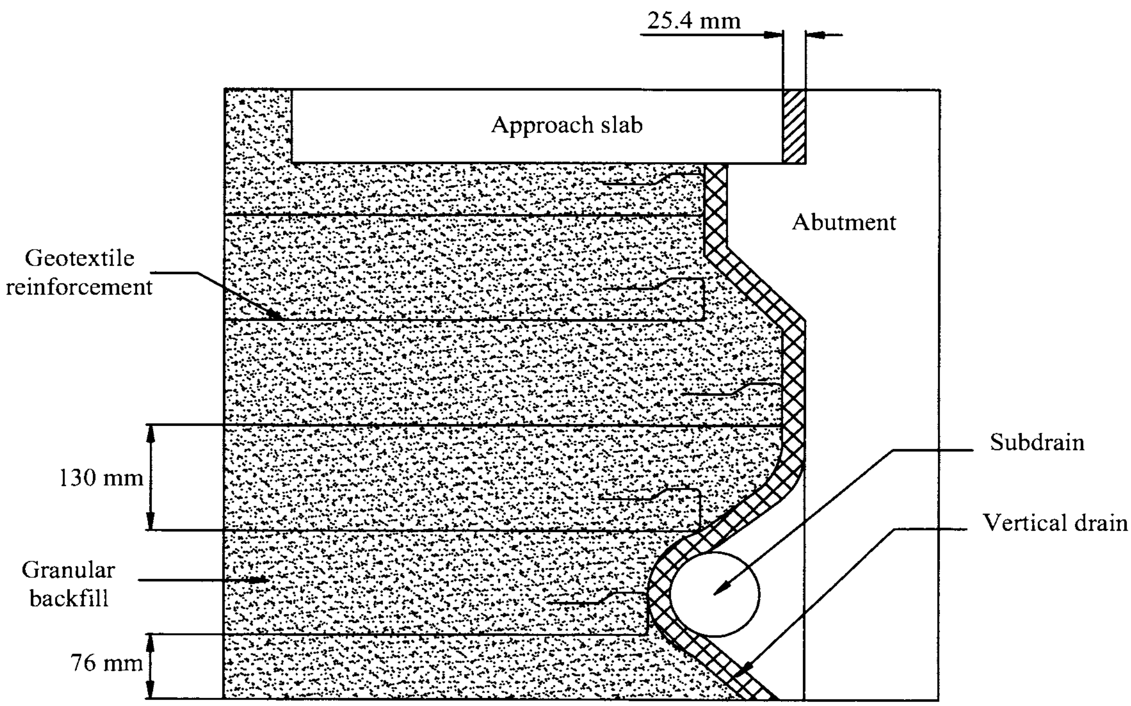
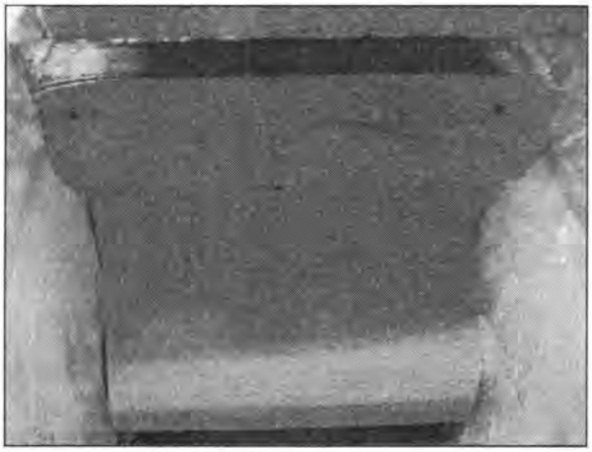


Figure 3.306 - Schematic of Test 7 Drainage Details



(a) Front View **(b) Side view**

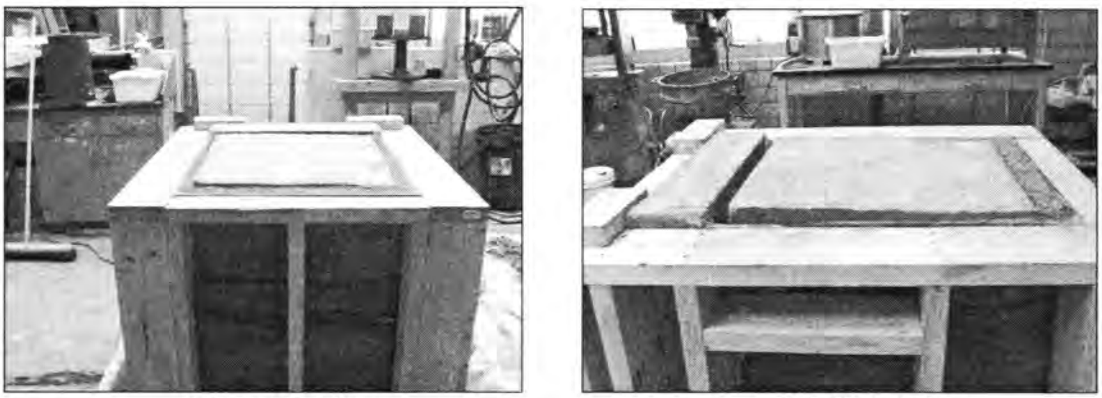
Figure 3.307 - Attaching the Geocomposite Drain to the Abutment

The water started to flow out of the subdrain after 4 minutes from starting the test. At the beginning of the test the water flowing out of the subdrain was clear indicating that the vertical drain stopped the fines from being washed out. However, after 30 minutes from starting the test, the granular backfill started passing through the plexi glass-vertical drain interface, and as a result some fines were observed being washed out. This problem is not expected to occur in the field because the vertical drain will be covering the wingwalls as well. Once the maximum steady state capacity of the vertical geocomposite drainage was exceeded, the water passed through the vertical drain fabric to the reinforced backfill which lead to the formation of a void and settlement. Table 3.16 summarizes the results of this test. The maximum measured settlement was 5.4 cm (See Figures 3.308 and 3.309). The maximum void developed was 13.9 cm at the abutment and extended 16.5 cm away from the abutment. Figure 3.310 shows the void developed after removing the approach slab. The maximum steady flow (222 cm^3/sec) increased by a factor of 3.5 compared to test 6.

However, when the maximum steady state flow of the vertical drain was exceeded, water started to overflow and saturate the backfill. As a result soil collapse and void development started to occur. The drainage capacity is not expected to be exceeded in the field since it was tested under extreme conditions.

Table 3.16 - Key Results from Test 7

Backfill type	Reinforced granular backfill with vertical drainage
Moisture content	4.2 %
Compaction	By own weight
Settlement (cm)	Left side: 5.1 cm
	Right side: 5.4 cm
Void (cm)	Left side: 12.7 cm deep at the abutment face and extending 24.1 cm under the approach slab
	Right side: 13.9 cm deep at the abutment face and extending 16.5 cm under the approach slab
Maximum steady state flow (cm^3/sec)	222
Time for water to drain (min)	4



(a) Front View (b) Side View

Figure 3.308 - Position of Approach Slab before Test 7

(a) Front View **(b) Side View**

Figure 3.309 - Position of Approach Slab after Test 7

Figure 3.310 - Void Developed under the Approach Slab

3.4.11 Test 8: Vertical Geocomposite Drainage System with Backfill Reinforcement (STRIPDRAIN 75)

The purpose of this test was to evaluate the performance of the granular backfill and the bridge approach using vertical drainage STRIPDRAIN 75. According to the manufacturer, granular, well graded backfill material is the most suitable backfill for this drain. The geocomposite drain consists of a HDPE polymer core that is 19 mm thick and laminated with a nonwoven, needle-punched geotextile. (See Appendix A for detailed specifications provided by the supplier). The setup of this test was similar to test 7 (See

Figure 3.311). Granular backfill was placed within the bulking moisture range (3.7%) and compacted by its own weight to simulate dumping of backfill material behind the abutment.

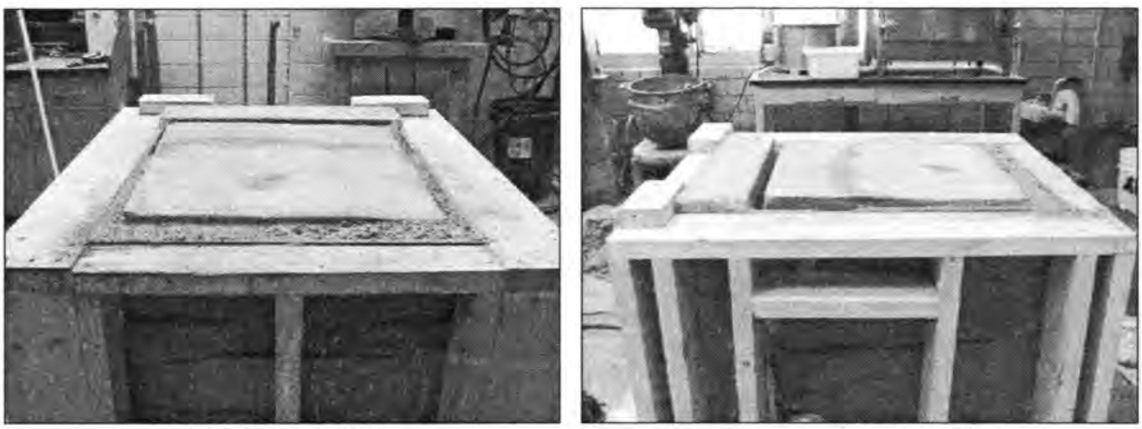
Figure 3.311 - Attaching the Geocomposite Drain to the Abutment

The water started to flow out of the subdrain after 1 minute from starting the test. No fines were washed out during the test. Similar to test 7, once the maximum steady state flow (383 cm^3/sec) of the geocomposite drain was exceeded, water started to overflow saturating the backfill, and as a result settlement and void development were observed. However, the maximum steady state flow is not expected to be exceeded in the field because it was tested under severe conditions. The maximum steady state flow was 72% higher for this vertical drain compared to the product used in test 7, and the void formed was approximately 70% smaller (See Table 3.17). The maximum approach slab settlement was 6.4 cm, which is approximately equal to the settlement observed at test 7 (See Figures 3.312 and 3.313). The void was 3.8 cm deep at the abutment and extended 11.4 cm away from the abutment; in addition, the void extended the full width of the approach slab (See Figure 3.314). Furthermore, drainage occurred as water rose from the bottom and filled the subdrain.

Overall the geocomposite drain is a good candidate to be used at bridge sites when combined with moisture control, and compaction of granular backfill.

Table 3.17 - Key Results from Test 8

Backfill type	Reinforced granular backfill
Moisture content	3.7 %
Compaction	By own weight
Settlement (cm)	Left side: 5.7 cm
	Right side: 6.4 cm
Void (cm)	Left side: 3.8 cm deep at the abutment face and extending 10.2 cm under the approach slab
	Right side: 3.8 cm deep at the abutment face and extending 11.4 cm under the approach slab
Maximum steady state flow (cm^3/sec)	383
Time for water to drain (min)	1



(a) Front View **(b) Side View**

Figure 3.312 - Position of Approach Slab before Test 8

(a) Front View (b) Side View

Figure 3.313 - Position of Approach Slab after Test 8

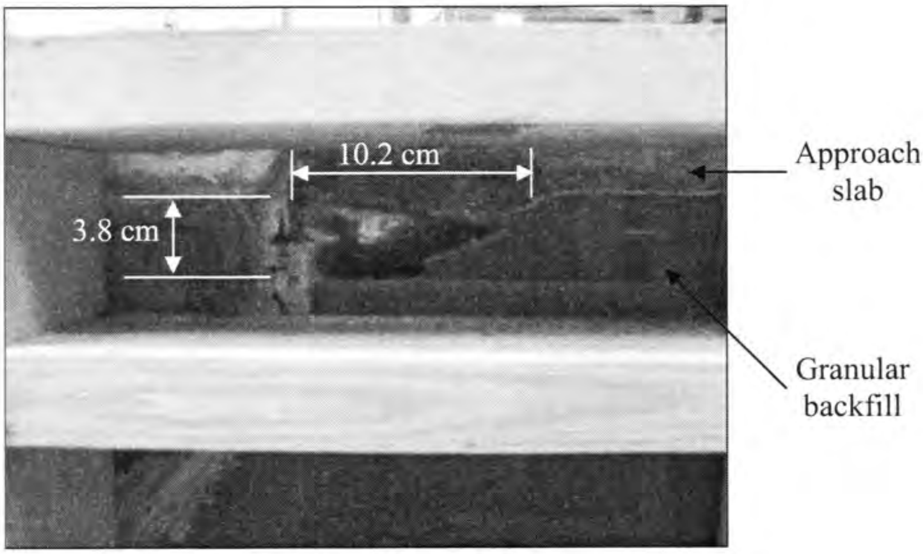


Figure 3.314 - Void Developed under the Approach Slab

3.4.12 Test 9: Granular Backfill with Tire Chips behind the Abutment

The purpose of this test was to evaluate the use of tire chips as a drainage material behind the abutment as well as its effectiveness in reducing the approach slab settlement and void development. A layer of tire chips, which was 180 mm wide, was placed behind the abutment without compaction. A 25.4 mm (1 in.) foam board was placed between the tire chips and the granular backfill. The granular backfill was placed within the bulking moisture content range (3.9%) and compacted by its own weight to simulate dumping of backfill material behind the abutment (See Figures 3.315 and 3.316).

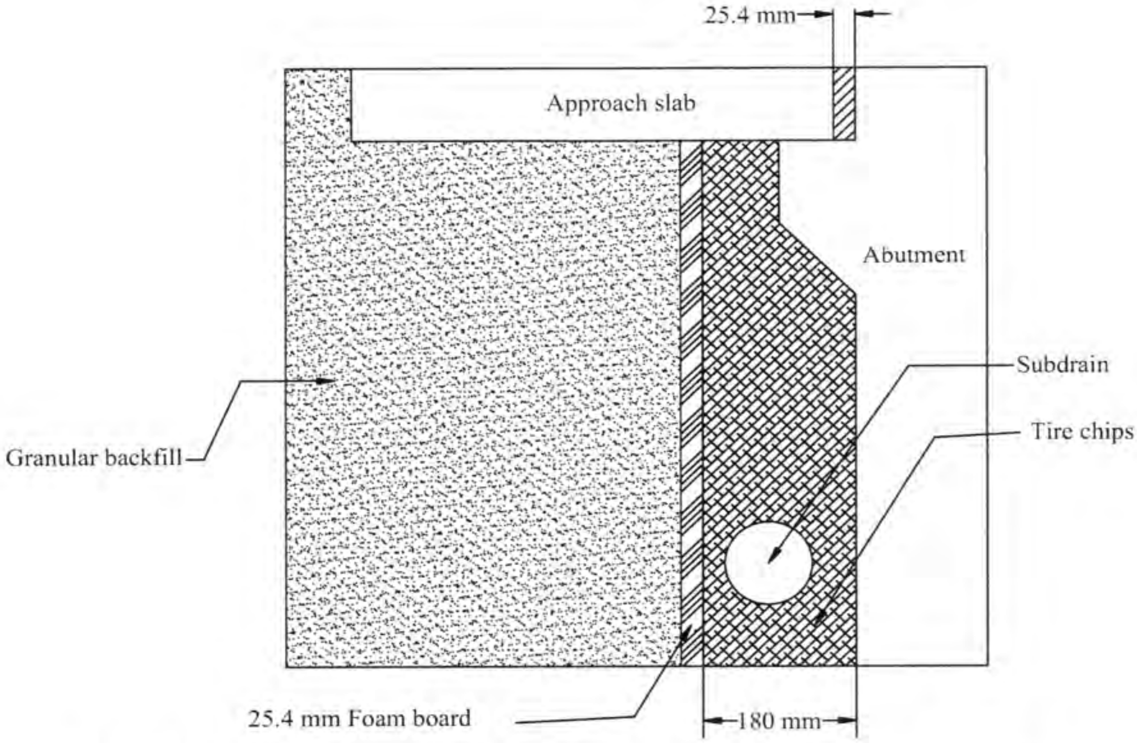


Figure 3.315 - Schematic of Test 9; Using Tire Chips behind the Abutment

Figure 3.316 - Drainage Detail for Test 9

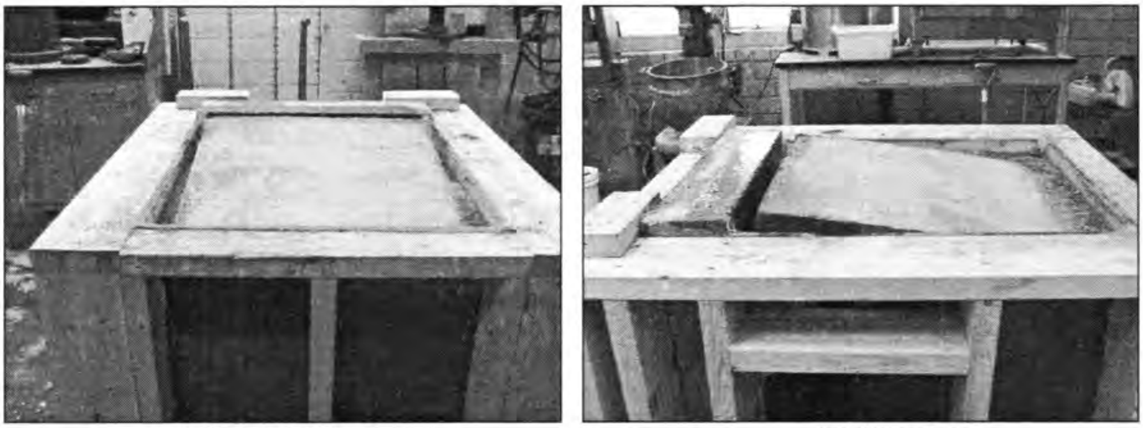
The water started to flow out of the subdrain after one minute from starting the test. The use of tire chips minimized the settlement and the void size but did not prevent them from occurring (See Table 3.18). The maximum void formed was 5.1 cm deep at the abutment face and extended 7.6 cm away from the abutment. The void was formed in the granular backfill after the foam board and was discontinuous (i.e. did not extend the full width of the approach slab). The maximum measured settlement was 4.8 cm which is 25% less than test 8 (See Figures 3.317 and 3.318). The steady state flow was 43% higher than test 8. In addition, water was drained from the top and bottom portions of the subdrain, which indicate that the tire chips did not block the openings at the top of the drainage pipe. Although 30% of the tire chips are smaller than the subdrain openings, none of the tire chips were washed out.

Table 3.18 - Key Results from Test 9

Backfill type	Granular backfill with tire chips as drainable material
Moisture content	3.9 %
Compaction	By own weight
Settlement (cm)	Left side: 4.8 cm
	Right side: 4.4 cm
Void (cm)	Left side: 2.5 cm deep at the foam edge and extending 5.1 cm under the approach slab
	Right side: 5.1 cm deep at the foam edge and extending 7.6 cm under the approach slab
Maximum steady state flow (cm^3/sec)	552
Time for water to drain (min)	1

(a) Front View (b) Side View

Figure 3.317 - Position of Approach Slab before Test 9



(a) Front View (b) Side View

Figure 3.318 - Position of Approach Slab after Test 9

3.4.13 Test 10: Using Tire Chips behind the Abutment with Soil Reinforcement

This test evaluated the effects of combining tire chips behind the abutment with mechanically reinforced granular backfill on the void size, approach slab settlement, and maximum steady state flow. A layer of tire chips, which is 180 mm wide, was used behind the abutment, and a 25.4 mm (1 in.) foam board was used to separate the granular backfill from the tire chips (i.e., similar to test 9). The first reinforcing geotextile layer was placed 76 mm from the bottom of the model. The following layers were placed every 130 mm lifts (See Figures 3.319 and 3.320). The granular backfill was placed within the bulking moisture content range (4.0%) and compacted by its own weight to simulate dumping of backfill material behind the abutment. The geotextile reinforcement was folded around the backfill at the foam board. The length of the folded geotextile was approximately 130 mm. The geotextile used for backfill reinforcement was CONTECH C-80NW (See Appendix A for specifications).

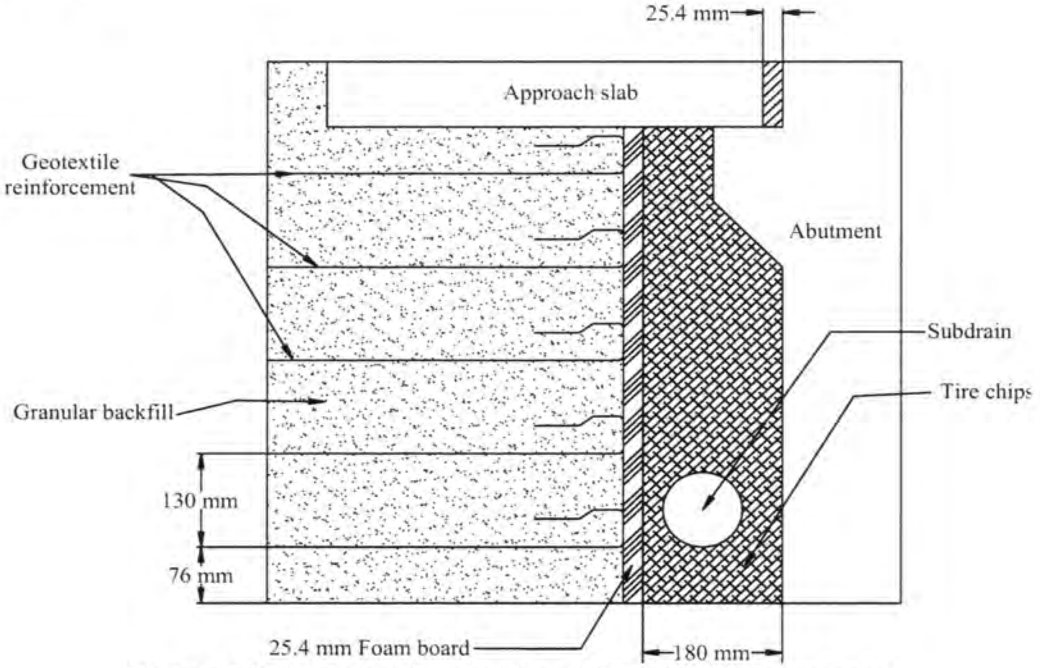


Figure 3.319 - Using Tire chips with Soil Reinforcement

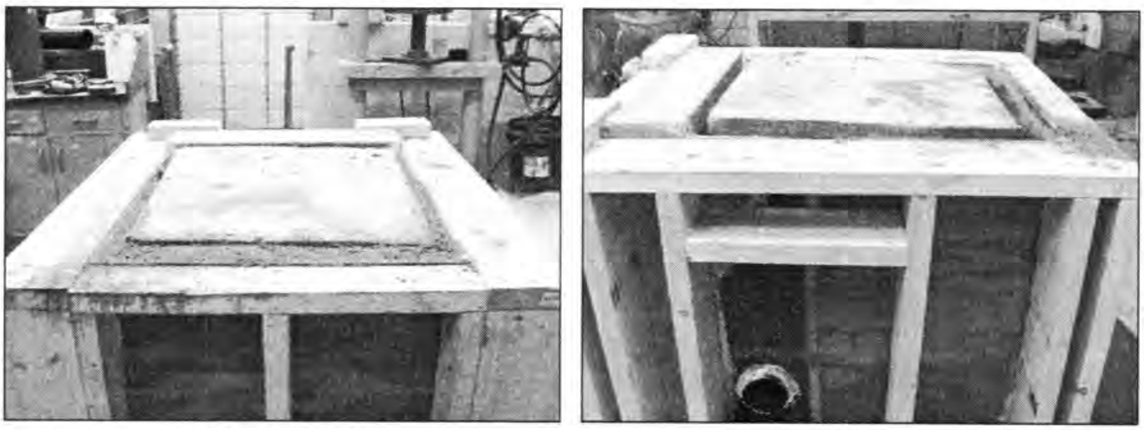
(a) Front View **(b) Side view**

Figure 3.320 - Foam Board Separating Tire Chips and Granular Backfill

The water started to flow out of the subdrain after one minute from starting the test. Using geotextile reinforcement decreased the settlement to 3.2 cm (See Figures 3.321 and 3.322) which is 33% less than the settlement measured in test 9 (See Table 3.19). Furthermore, using geotextile reinforcement prevented the formation of the void under the approach slab. A maximum steady state flow of 554 cm^3/sec was achieved, which is approximately equal to test 9. Drainage occurred from both the top and the bottom of the subdrain indicating no plugging of subdrain openings.

Table 3.19 - Key Results from Test 10

Backfill type	Reinforced granular backfill with tire chips as drainable material
Moisture content	4.04 %
Compaction	By own weight
Settlement (cm)	Left side: 2.5 cm
	Right side: 3.2 cm
Void (cm)	None
Maximum steady state flow (cm^3/sec)	554
Time for water to drain (min)	1



(a) Front View **(b) Side View**

Figure 3.321 - Position of Approach Slab before Test 10

(a) Front View **(b) Side View**

Figure 3.322 - Position of Approach Slab after Test 10

3.4.14 Test 11: Using Porous Backfill

The purpose of this test was to evaluate the use of porous fill as a substitute for granular backfill, and its effect on steady state flow, approach slab settlement, and void formation. This drainage detail was chosen based on the good performance of porous fill (pea gravel) in the collapse index test. The pea gravel was placed at a moisture content of 4.59%, and compacted by its own weight (See Figures 3.323 and 3.324).

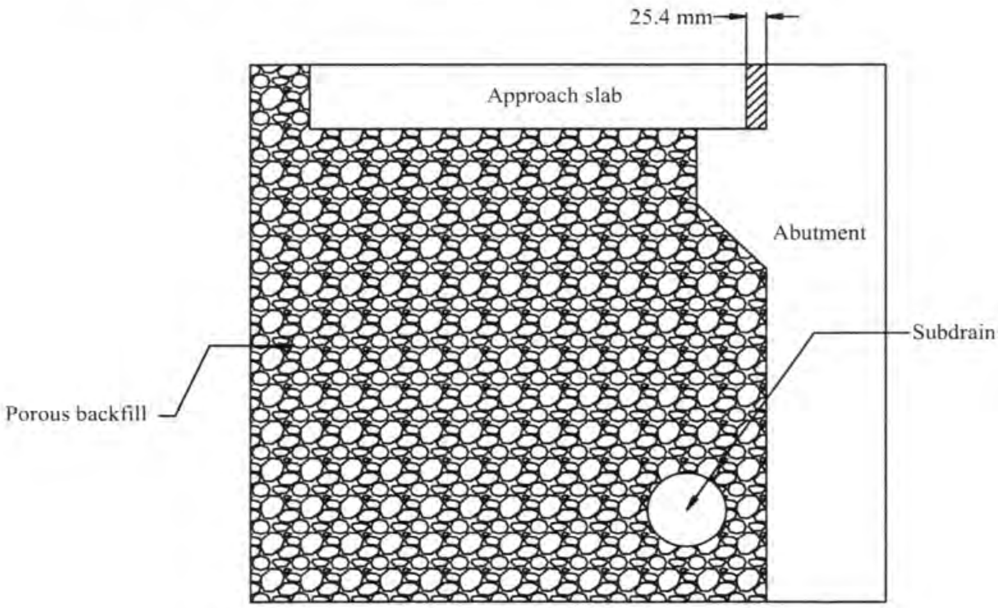


Figure 3.323 - Drainage Detail for Test 11

(a) Front View

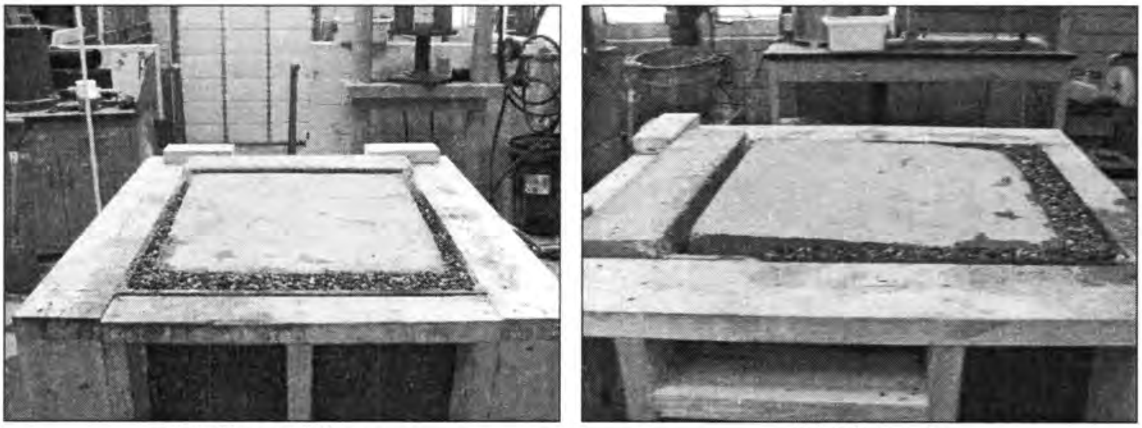
(b) Side View

Figure 3.324 - Placing Porous Fill behind the Abutment

Water started to flow out of the subdrain after 4 minutes from starting the test. The pea gravel included some fines which were washed out. The water started to clear, indicating no more fines being washed out, after 18 minutes from starting the test. Table 3.20 summarizes the results of this test. The porous fill prevented the void formation and approach slab settlement completely (See Figures 3.325 and 3.326). The maximum steady state flow was 92 cm^3/sec, which was approximately 3 times higher than the maximum steady state flow measured using the current Iowa DOT specification drainage detail. However, the maximum steady state flow was lower than tests using geocomposite drains and tire chips. Drainage occurred as the water rose from the bottom and filled the subdrain. Despite of the relatively low flow, this drainage detail can be applied at bridge sites due to its good performance and simple construction sequence.

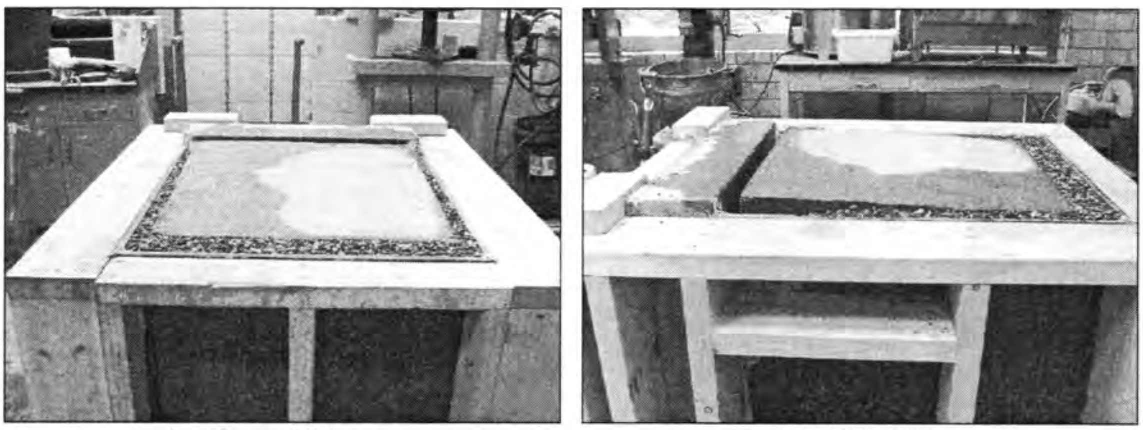
Table 3.20 - Key Results from Test 11

Backfill type	Porous fill
Moisture content	4.59 %
Compaction	By own weight
Settlement (cm)	No settlement
Void (cm)	None
Maximum steady state flow (cm^3/sec)	92.0
Time for water to drain (min)	4



(a) Front View **(b) Side View**

Figure 3.325 - Position of Approach Slab before Test 11



(a) Front View **(b) Side View**

Figure 3.326 - Position of Approach Slab after Test 11

3.4.15 Discussion

The water management bridge approach model demonstrated that Iowa DOT current drainage design, with granular backfill placed without compaction and within the bulking moisture content range, would results in a large void, settlement, and low drainage capacity. The same Iowa DOT drainage design but with saturated granular backfill does not experience settlement or void development; however, the drainage capacity is still low. The largest void and settlement were observed in the test replicating the field practices. Furthermore, using STRIPDRAIN 75 increased the drainage capacity to 383 cm^3/sec and reduced the void by 7.6 cm when compared to current Iowa DOT design. The use of tire chips behind the abutment showed the highest drainage capacity of 552 cm^3/sec and reduced the void size by 6.4 cm when compared to current Iowa DOT design. Using porous backfill as a substitute for granular backfill prevents approach slab settlement and void development, and increase the drainage capacity by three times when compared to current Iowa DOT design. Finally, using backfill reinforcement does not prevent void development, decrease drainage capacity, and decrease the settlement. Table 3.21 summarizes the results obtained from all tests.

Table 3.21 - Summary of Water Management Bridge Approach Tests

Description	Settlement (cm)	Void (cm)	Maximum flow rate (cm^3/sec)	Time for water to drain (min)
1. Iowa DOT Design (w = 3.0%)	5.1	11.4	32	10
2. Iowa DOT Design (w = 12.6%)	None	None	31	12
3. Field Practice-1	5.7	10.2	33.5	10
4. Field Practice-2	5.7	5.1	67	11
5. Wrapping the Porous Fill with geotextiles	5.1	6.4	82	10
6. Geotextile around Porous fill and Backfill reinforcement	2.5	4.4	63	7
7. Geocomposite Drain and backfill reinforcement (Tenax Ultra-Vera™)	5.4	12.7	222	4
8. Geocomposite Drain and backfill reinforcement (STRIPDRAIN 75) w = 3.7%	6.4	3.8	383	1
9. Tire chips behind the abutment (w = 3.9%)	4.8	5.1	552	1
10. Tire chips with backfill reinforcement	3.2	None	554	1
11. Porous backfill	None	None	92	4

3.5 Characterization of Bridge Approach Settlement

One of the objectives of this research is to characterize the bridge approach settlement in order to recommend a threshold limit beyond which maintenance is required. This is done by evaluating bridge approach profiles, International Roughness Index (IRI) data, Iowa DOT ratings, and the approach slab rating system developed by Louisiana Transportation and Research Center (LTRC).

The Iowa DOT ratings for 26 bridge approaches on U.S. 65 were done by evaluating the ride quality of bridge approaches using a 30,000 lbs truck driving at a speed of 65 to 68 mph over approach slabs. The rating was done according to how severe a bump was felt at the two ends of the bridge.

The rating system developed by LTRC was a modification from the LTRC IRI pavement evaluation ratings (See Table 3.22). To evaluate an approach slab, the highest IRI value collected was used to rate the performance of the approach slab (Das *et al.* 1999).

Table 3.22 - Refined IRI Approach Slab Rating System Developed by LTRC (Das *et al.* 1999)

Range (IRI), m/km	Rating
0.0 to 3.9	Very Good
4.0 to 7.9	Good
8.0 to 9.9	Fair
10.0 to 11.9	Poor
12.0 and above	Very Poor

3.5.1 International Roughness Index (IRI)

The IRI data, obtained from Center for Transportation Research and Education (CTRE), was measured in 2003. The IRI data was provided for 20 bridges on U.S 65. The data provided was used to plot IRI graphs for 26 of the 40 bridge approaches. A sample of such graphs is shown in Figure 3.327. The remaining graphs are shown in Appendix B. The transition between the roadway and the approach slab and the transition between the approach slab and the bridge have significantly higher IRI values. These values ranged from 3.9 to 11.8 m/km.

To check whether the higher IRI values observed at the transition between the road and the approach slab and the transition between the approach slab and the bridge was caused by approach slab settlement, the 2001 IRI values of the bridges on U.S. 65 were compared to the 2003 data. A sample of the developed graphs, shown in Figure 3.328, indicate that the high IRI values at the two transition locations increase with time which points out that the high roughness values observed are a function of approach slab settlement. Therefore, the IRI values were used as a criteria to rate the bridge approach performance.

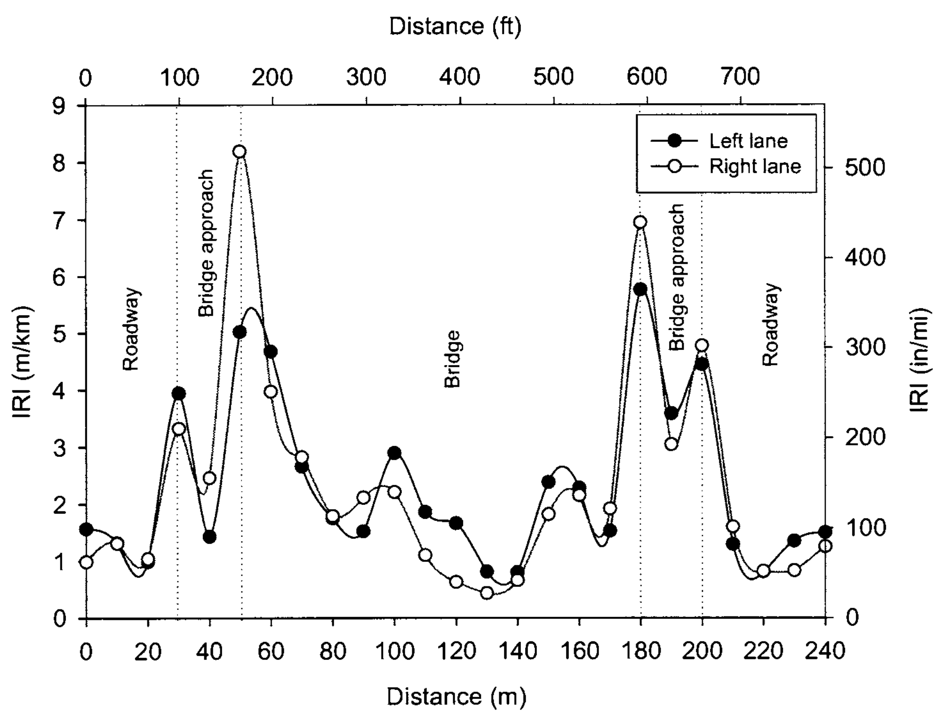


Figure 3.327 - IRI Graph (Bridge No. 7777.0065 SBL)

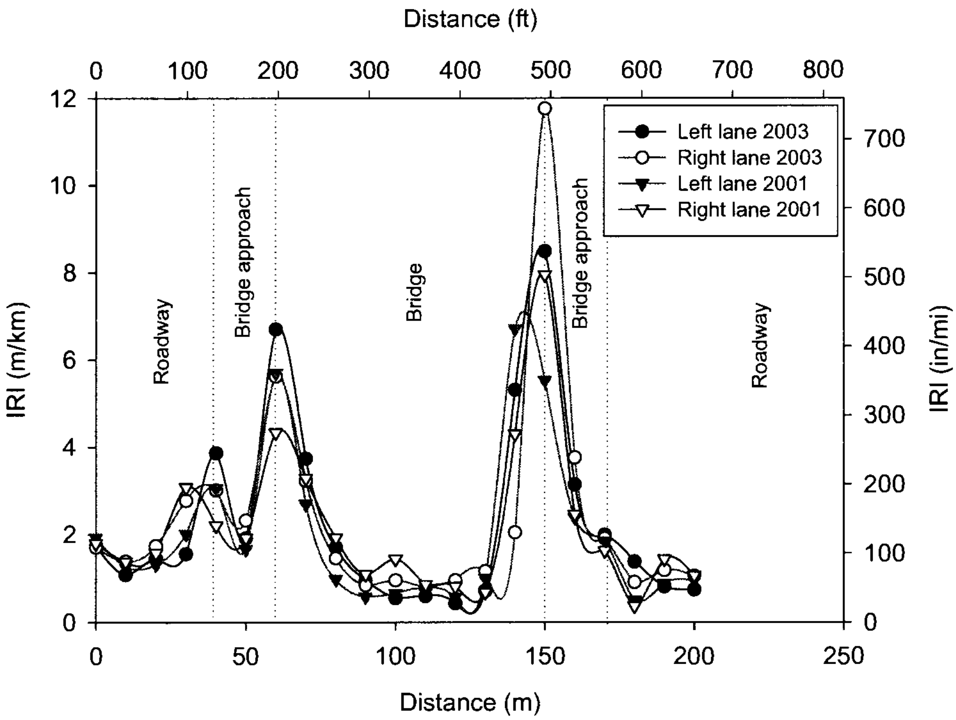


Figure 3.328 - Increase of International Roughness Index with Time (Bridge No. 7773.0065 NBL)

3.5.2 Bridge Approach Profiles

The profiles for 13 bridge approaches on U.S. 65 were generated by measuring the elevation of the approach slab relative to the elevation of the bridge deck. The data was collected at the edge line of the right lane. To evaluate the performance of the bridge approaches, The Bridge Approach Index (BI), which is defined as the area between the original profile and the existing profile of the approach slab divided by the approach slab length, was used. The area was determined by subtracting the integration of the original profile, which is assumed to be a straight line connecting the roadway to the bridge deck, and the existing profile over the length of the approach slab. The higher the calculated BI the worse the approach slab condition. For example, Figure 3.329 shows the settlement of two approach slabs relative to the original profile where the north end settled more than the south end, which is reflected in the higher area calculated between the original and existing profile.

The area calculated for the north end was 4.66 m^2 (50.16 ft^2), while the area calculated for the south end was 2.63 m^2 (28.43 ft^2). The BI values for these two approaches were 0.25 m and 0.17 m respectively.

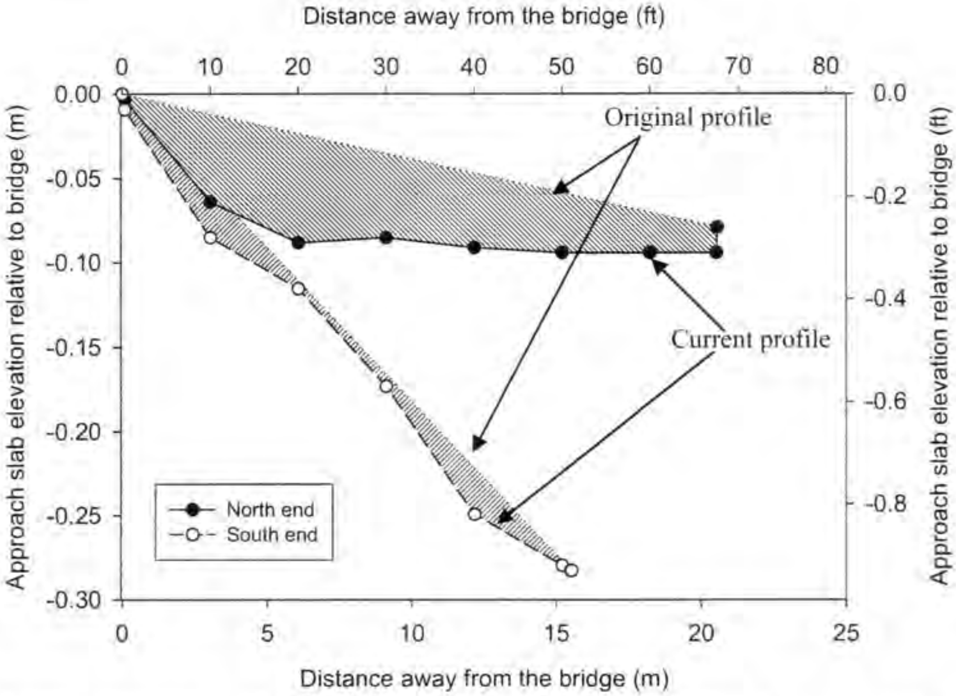


Figure 3.329 - Elevation of Approach Slab Relative to Bridge (Bridge No. 7777.0065 NBL)

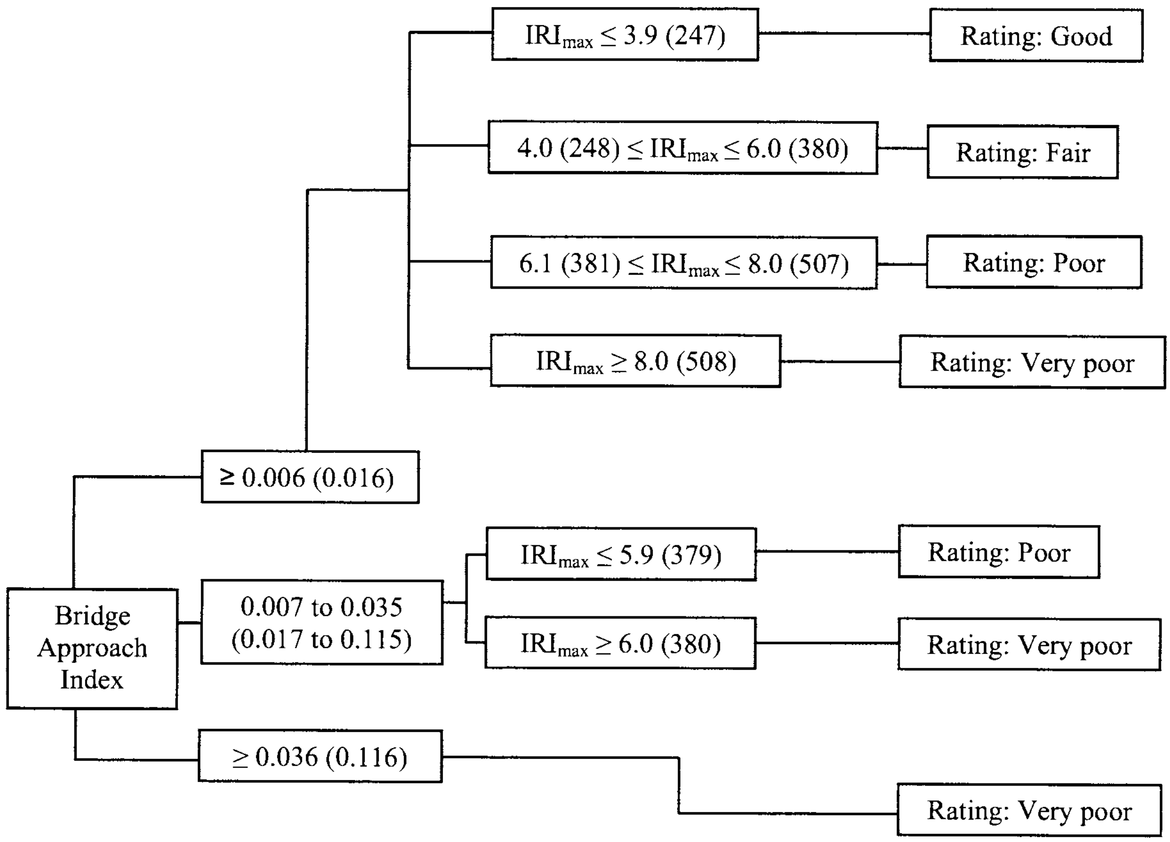
3.5.3 Rating Criteria

Table 3.23 presents a summary of the LTRC ratings, BI values, and IRI data used to develop the rating criteria for bridge approaches. The table also presents the Iowa DOT ratings of the approach slabs which were generally higher than the LTRC ratings indicating that the rating criteria provided by the LTRC (See Table 3.22) is not applicable for approach slabs in Iowa. In addition, using only IRI values does not adequately assess the performance of bridge approaches. Therefore, the author suggests a rating system for characterizing approach slab performance based on the BI, which estimates the approach slab settlement due to settlement of the foundation soil and/or compression of the embankment material, and the maximum IRI value at the bridge approach, which estimates the settlement caused by the bump at the end of the bridge. Figure 3.330 shows the developed rating system.

According to the new developed rating system for U.S. 65 bridge approaches (See Table 3.24), 84% of the approach slabs were rated very poor, 8% poor, 4% fair, and 4% good. Approach slabs rated below fair require maintenance; and thus 92% of the bridge approaches on U.S. 65 require maintenance.

Table 3.23 - Summary of Data used to Rate the Performance of Approach Slabs

Bridge no.	Location	IRI_{max} (m/km)	IADOT rating	LTRC Rating	Area (m^2)	Length (m)	BI (m)
7773.0065	NE-NBL	11.8	Poor	Poor	0.0092	15.35	0.0006
7773.0065	SE-NBL	6.7	Good	Good	0.69	23.16	0.029
7774.0L065	SE-SBL	8.3	Poor	Fair	0.65	19.15	0.034
7776.8065	NE-NBL	8.1	Poor	Fair	0.39	15.5	0.025
7776.8065	SE-NBL	5.8	Fair	Good	0.93	18.69	0.049
7776.8065	NE-SBL	5.4	Poor	Good	2.41	18.69	0.129
7776.8065	SE-SBL	5.6	Fair	Good	0.11	18.54	0.0059
7777.0065	NE-NBL	6.7	Poor	Good	4.64	18.54	0.25
7777.0065	SE-NBL	5.3	Poor	Good	2.63	15.5	0.169
7778.1065	NE-NBL	6.8	Poor	Good	0.39	14.29	0.027
7778.1065	SE-NBL	5.7	Poor	Good	0.05	20.27	0.0024
7778.1065	NE-SBL	10.3	Poor	Poor	2.75	21.28	0.129
7779.4065	NE-NBL	8.5	Poor	Fair	0.013	17.63	0.0007
7779.4065	SE-NBL	9.9	Poor	Fair	17.49	18.85	0.928
7779.4065	NE-SBL	7.7	Fair	Good	0.58	18.7	0.031
7779.4065	SE-SBL	4.8	Poor	Good	5.51	18.24	0.302
7780.8R065	NE-NBL	7.9	Poor	Good	1.61	16.57	0.097
7781.2065	NE-NBL	11.6	Very poor	Poor	3.08	23.71	0.129
7781.2065	SE-NBL	7.6	Very poor	Good	5.51	16.41	0.336
7781.2065	NE-SBL	9.6	Very poor	Fair	0.55	17.18	0.032
7781.2065	SE-SBL	8.5	Very poor	Fair	0.89	17.18	0.052
7782.8L065	NE-SBL	3.9	Poor	Very Good	0.48	15.81	0.030
7783.1065	NE-NBL	5.8	Poor	Good	1.66	16.42	0.101
7783.1065	SE-NBL	5.2	Fair	Good	14.03	20.06	0.699
7783.1065	NE-SBL	3.8	Poor	Very Good	0.021	18.24	0.0011
7783.1065	SE-NBL	5.5	Poor	Good	25.17	15.2	1.656



*Approach Slab Index, m (ft)
*IRI, m/km (in/mi)

Figure 3.330 - Proposed Rating System for Bridge Approach Performance

Table 3.24 - U.S. 65 Bridge Approach Ratings According to the New Developed Rating System

Bridge no.	Location	Rating
7773.0065	NE_NBL	Very poor
7773.007	SE_NBL	Very poor
7774.0L065	SE_SBL	Very poor
7776.807	NE_NBL	Very poor
7776.807	SE_NBL	Very poor
7776.807	NE_SBL	Very poor
7776.807	SE_SBL	Poor
7777.007	NE_NBL	Very poor
7777.007	SE_NBL	Very poor
7778.107	NE_NBL	Very Poor
7778.107	SE_NBL	Fair
7778.107	NE_SBL	Very poor
7779.407	NE_NBL	Very poor
7779.407	SE_NBL	Very poor
7779.407	NE_SBL	Very poor
7779.407	SE_SBL	Very poor
7780.8R065	NE_NBL	Very poor
7781.207	NE_NBL	Very poor
7781.207	SE_NBL	Very poor
7781.207	NE_SBL	Very poor
7781.207	SE_SBL	Very poor
7782.8L065	NE_SBL	Poor
7783.107	NE_NBL	Very poor
7783.107	SE_NBL	Very poor
7783.107	NE_SBL	Good
7783.107	SE_NBL	Very poor

3.5.4 Discussion

From the IRI data it is observed that the highest IRI values are at the transition between the roadway and the approach slab, and the transition between the approach slab and the bridge. These values ranged from 3.8 m/km to 11.8 m/km. IRI values at the bridge approach increased with time indicating continuous settlement with time.

The BI together with the maximum IRI value at the approach slab are used to rate the performance of the approach slab and initiate maintenance. According to the new developed rating system, 84% of the approach slabs on U.S. 65 are rated very poor, 8% poor, 4% fair, and 4% good. Bridges rated below fair require maintenance.

CHAPTER 4 - SUMMARY AND CONCLUSIONS

4.1 Summary

Bridge approach settlement and the formation of the bump at the end of the bridge affect about 25% of the bridges nationwide. Iowa DOT personnel believe that the problem in Iowa is more substantial than the national average; however, no criteria exist to initiate maintenance action. This study was undertaken to identify the factors contributing to bridge approach settlement. The investigation included: (1) a literature review and documentation of design, and construction and maintenance practices used by Iowa DOT and other state DOTs; (2) field inspection of existing and under construction bridges in all Iowa districts; (3) Monitoring and documentation of current maintenance practices; (4) characterization of backfill materials used behind bridge abutments; (5) modeling bridge approach sections for optimum drainage detail; and (6) characterization of bridge approach settlement using the International Roughness Index (IRI). During this study it was revealed that water management and backfill characteristics are two major reasons related to bridge approach settlement. The effects of these two factors on the behavior of approach slab sections are presented in this study.

4.1.1 Relevant Literature

Bridge approach settlement can be caused by: (1) seasonal temperature change; (2) loss of backfill material by erosion; (3) poor construction practices (i.e., poor joints, poor drainage, poor compaction, and erodible backfill material); (4) settlement of foundation soil; and (5) high traffic loads. However, the two primary causes reported in the literature are the lateral movement of the bridge and the embankment settlement.

Seasonal ambient temperature cycles between summer and winter and the corresponding movement of the bridge superstructure and the abutment in the case of integral bridges displaces the soil behind the abutment which creates a void. Once a void is created, erosion and loss of backfill material are expected to occur. In an attempt to prevent this from happening, researchers and other states DOTs recommended several changes, which include:

1. Connecting the approach slab to the bridge, reducing the joint width, and using new joint sealers. A documentation of the practices in 37 states revealed that 32% of them tie the bridge approach to either the bridge deck or the bridge abutment in case of integral abutments and 57% in case of non-integral abutment. Current Iowa DOT specifications tie the approach slab to the abutment in case of non-integral abutments only. Connecting the bridge approach to the abutment minimizes the change of the joint width as a result of seasonal temperature change which helps keep the joint sealed and prevent the water from flowing to the embankment soil. Furthermore, it eliminates the potential of the approach slab to slide off the paving notch as a result of bridge movement. The joint width used in 12 states varied from 1.3 cm to 5.1 cm; however, a 10.2 cm expansion joint is used in Iowa. Other states used a v-shaped rubber gland as a new joint sealing system which improved the performance of the joint at the end of the bridge.
2. Using compressible elastic material behind the abutment to reduce the effects of abutment movement on the surrounding soil.
3. Using geosynthetic reinforced backfill and geotextiles around porous backfill to reduce erosion. This design creates stiffer backfill around the bridge which help in decreasing the settlement difference between the bridge and the surrounding soil. To achieve this, other states use shallow foundations to support the bridge abutments.
4. Using a geocomposite drainage system around the abutment to prevent erosion and loss of backfill material, and for a higher drainage capacity

4.1.2 Field Investigations

Field observations of existing bridges revealed that 26% of the expansion joints of the inspected bridges were not properly sealed, allowing water to flow down the bridge embankment. The Iowa DOT uses two types of material to fill the expansion joints – flexible foam and recycled tires. Generally, recycled tires performed better than flexible foam. Moreover, the maximum change of the joint width monitored for 15 months was 2 cm which indicates that the joint is wider than necessary and can be reduced.

Inadequate drainage indicated by void development under the approach slab, dry subdrains behind bridge abutments, ponding of water on bridge embankments, and/or erosion around the bridge was observed at 63% of the inspected bridges. The bridge end drain detail observed at bridge no. 5596.2S169 at district 2 showed better performance than other bridge end drain details, where no erosion was observed around the bridge. Erosion of the bridge embankment was observed at 36% of the visited bridges, while a void was formed at approximately 20% of these bridges.

Field visits for nine under construction bridges revealed that non compacted granular backfill, classified as SP, was used as a backfill material at moisture contents ranging from 3% to 6% and that porous backfill was not used around the subdrain. Out of the nine inspected bridges, porous backfill around the subdrain was observed at two bridge sites only.

Monitoring and documenting maintenance practices in Iowa showed that the common practice to reduce bridge approach settlement is resurfacing using hot mixed asphalt. Grouting behind bridge abutments to fill the voids developed was another common practice which causes stresses to develop at the bridge deck resulting in severe damage. Both methods are temporary solutions since settlement occurs shortly after maintenance as well as being a continuous expense. Replacement of the bridge approach occurs when the slab is significantly damaged as in the case of faulting; however, the replacement is not accompanied by proper replacement and compaction of new backfill material and thus settlement is deferred but not prevented. The URETEK Inc. maintenance method, which consists of injecting expansive polyurethane under the approach slab to lift it to its original position, successfully raised the approach slab and reduced the settlement. Future monitoring of the approach slabs maintained by URETEK Inc. is needed to observe whether further settlement, void formation, and erosion are prevented.

4.1.3 Characterization of Backfill Materials

Comparing the perforated pipe opening to five granular backfill gradations collected from various bridge sites revealed that on average 72% of this material can pass through the pipe openings.

The collapse index of backfill materials is determined by measuring the change of the material volume as the moisture content increase. It is found that granular material experience a maximum collapse of 6% at moisture contents in the range of 3% to 6%. This range of moisture contents, which is within the bulking moisture content range, is similar to the moisture contents measured in the field. Porous backfill material does not experience any collapse at various moisture contents.

Another property of granular backfill materials used behind bridge abutments is its erodibility. The gradation range specified by Iowa DOT for granular backfill includes 20% to 100% passing the No. 8 sieve. Part of this wide range is included within the range of most erodible soils. The gradation range specified by the Iowa DOT for porous backfill materials does not fall within the range of most erodible soils.

Furthermore, the drainage properties of backfill materials were tested using the water management bridge approach model. Eleven models using granular and porous backfill materials with geocomposite drains, tire chips, and geotextile reinforcement were tested. The maximum steady state flow rate achieved, the differential settlement, and the void size under the approach slab were monitored. The worst performance (minimum flow, maximum void, and maximum differential settlement) was observed for the model representing the field practices. Using porous backfill prevented the settlement and the void formation, and increased the flow capacity from 32 cm^3/sec to 92 cm^3/sec. Using tire chips reduced the settlement by 37%, prevented the void formation, and increased the flow to 554 cm^3/sec when compared to the test representing Iowa DOT drainage detail.

4.1.4 Characterization of Bridge Approach Settlement

To quantify the approach slab performance and establish a threshold to initiate maintenance, IRI data and the profiles of bridge approaches on U.S. 65, where Iowa DOT proposed a major maintenance project, were used to develop the rating criteria. The profiles of bridge approaches were used to calculate the Bridge Approach Index (BI), which is defined as the area between the current bridge approach profile and the original profile divided by the bridge approach length. The maximum value of IRI around the bridge and the BI are combined and used to develop the rating criteria. The new rating system shows that 92% of the inspected bridges on U.S. 65 require maintenance.

4.2 Conclusions

4.2.1 Literature Review

- Many states tie the approach slab to the abutment to keep the expansion joint sealed and its width constant.
- The expansion joint widths specified by 12 states are 50% to 87% smaller than the expansion joint width specified by Iowa DOT.
- Twenty states, including Iowa, specify the percentage of maximum dry density to be achieved in the field according to AASHTO T-99. The Moisture Density Relations of Soil Using a 2.5 kg Rammer and 305 mm Drop for Field Compaction.

- Many states use a v-shaped rubber gland which, according to the literature, completely seals the expansion joint. Therefore, it is expected to perform better than flexible foam and recycled tire materials used in Iowa.
- Using compressible elastic material behind the abutment reduces the effect of abutment movement on surrounding soil.

4.2.2 Field Investigations

- The maximum change in joint width due to bridge movement did not exceed 2 cm, which validates the reduction of the expansion joint width.
- Inadequate drainage around existing brides had a major role in the formation of the bump at the end of the bridge.
- The bridge end drain observed at bridge number 5596.2S169 at district 2 is more effective than other bridge end drain details observed.
- Compaction of granular backfill to the specified density was not conducted, and porous backfill around subdrains was not placed.
- The granular backfill was placed within the bulking moisture range where the collapse potential and the resistance to compaction are highest.
- Resurfacing approach slabs and grouting voids behind bridge abutments are a common maintenance practices in Iowa, but they do not necessarily solve the settlement problem.
- The URETEK Inc. maintenance method successfully lifted the approach slab to its original level. However, future monitoring is required to detect whether further settlement and erosion are prevented, and examine if stresses will develop at the bridge structure.

4.2.3 Characterization of Backfill Materials

- On average, 72% of the granular backfill materials used behind bridge abutments are smaller than the subdrain openings.
- Upon saturation, poorly compacted granular backfill experience 6% collapse at moisture contents ranging from 3% to 6%, while porous backfill does not collapse.
- Due to the wide range specified by Iowa DOT for the percent passing the No. 8 sieve (20%-100%) for granular backfill, the material is regarded as an erodible soil according to the chart provided by Briaud *et al.* (1997). While porous backfill materials are not erodibile according to the same chart.
- The water management bridge approach model representing the field practice showed the worst performance, while the model with porous backfill prevented the settlement, void formation, and increased the maximum steady flow from 32 cm^3/sec to 92 cm^3/sec.
- The water management bridge approach model where tire chips were used reduced the settlement by 37%, prevented the void formation, and increased the maximum steady flow to 554 cm^3/sec.

4.2.4 Characterization of Bridge Approach Setttlement

- To develop a rating system for approach slabs performances, the BI and the maximum IRI at the approach slab were used. According to the new rating system, 92% of the bridges on U.S. 65 require maintenance.

CHAPTER 5 - RECOMMENDATIONS

Based on the findings of this study the following recommendations are proposed to improve water management and backfill characteristics, and reduce bridge approach settlement.

5.1 New Bridges

- Reduce the expansion joint to a construction joint with a 25.4 mm width, and change the 'CF' joint to an expansion joint ('E' joint) with a width of 50.8 mm.
- The new expansion joint shall be sealed using a v-shaped rubber gland (See Figure 5.1).
- Bridge end drain observed at bridge number 5596.2S169 at district 2 shall be used for newly constructed bridges (See Figure 5.2).
- Connect the approach slab to the bridge abutment.
- A square abutment shall be constructed to eliminate the difficulties of forming the paving notch as well as eliminating the difficulty in compacting backfill material in the region surrounding the paving notch.
- Moisture content ranging from 8% to 12% for granular backfill during placement behind bridge abutments shall be specified to allow for easier compaction and reduction of soil collapse.

The above recommendations shall be combined with one of the following drainage details:

1. Using porous backfill behind the abutment (See Figure 5.3). This design option is simple to construct, increases the drainage capacity, and prevents settlement and void formation.

2. Using geocomposite vertical drainage system behind the abutment (See Figure 5.4). This drainage option has simple construction sequence, increases the drainage capacity, and reduces the void size and differential settlement.

3. Using geocomposite vertical drainage system behind the abutment with reinforced granular backfill (See Figure 5.5). This drainage detail has a more complex construction sequence compared to option 2; however, the differential settlement and the void size are further reduced due to the stiffer backfill.

4. Using tire chips behind the abutment (See Figure 5.6) which provides an elastic zone behind the abutment. The elastic region allows for lateral movement of the abutment as the temperature change without affecting the backfill. Furthermore, it has a very high drainage capacity.

5.2 Bridge Approach Maintenance

- The new developed rating system shall be used to evaluate the performance of bridge approach sections. Bridge approaches rated below fair require maintenance.
- The URETEK Inc. maintenance method shall be used to lift approach slabs to their original level, and the expansive polyurethane shall fill the voids developed behind bridge abutments.
- If replacement of the approach slab is necessary, then it shall be accompanied by replacement of the backfill according to the new parameters (i.e. saturated backfill and field density as determined by the relative density factor); in addition to installation of one of the recommended drainage details.

5.3 Future Research

Future research is needed to evaluate the recommended changes in bridge approach design. This shall be accomplished by full scale pilot projects where the performance of bridge approach sections are evaluated and compared under the effect of the suggested changes.

Further monitoring of bridge approaches maintained by URETEK Inc. is essential to identify any potential problem that may result from injecting polyurethane behind bridge abutments and to observe if settlement, erosion, and void development at bridge approach sections are prevented.

Continue monitoring of replaced and resurfaced bridge approaches on U.S. 65. This will improve our understanding of how fast settlement takes place at a newly maintained bridge as well as the life span of asphalt resurfaced bridge approach.

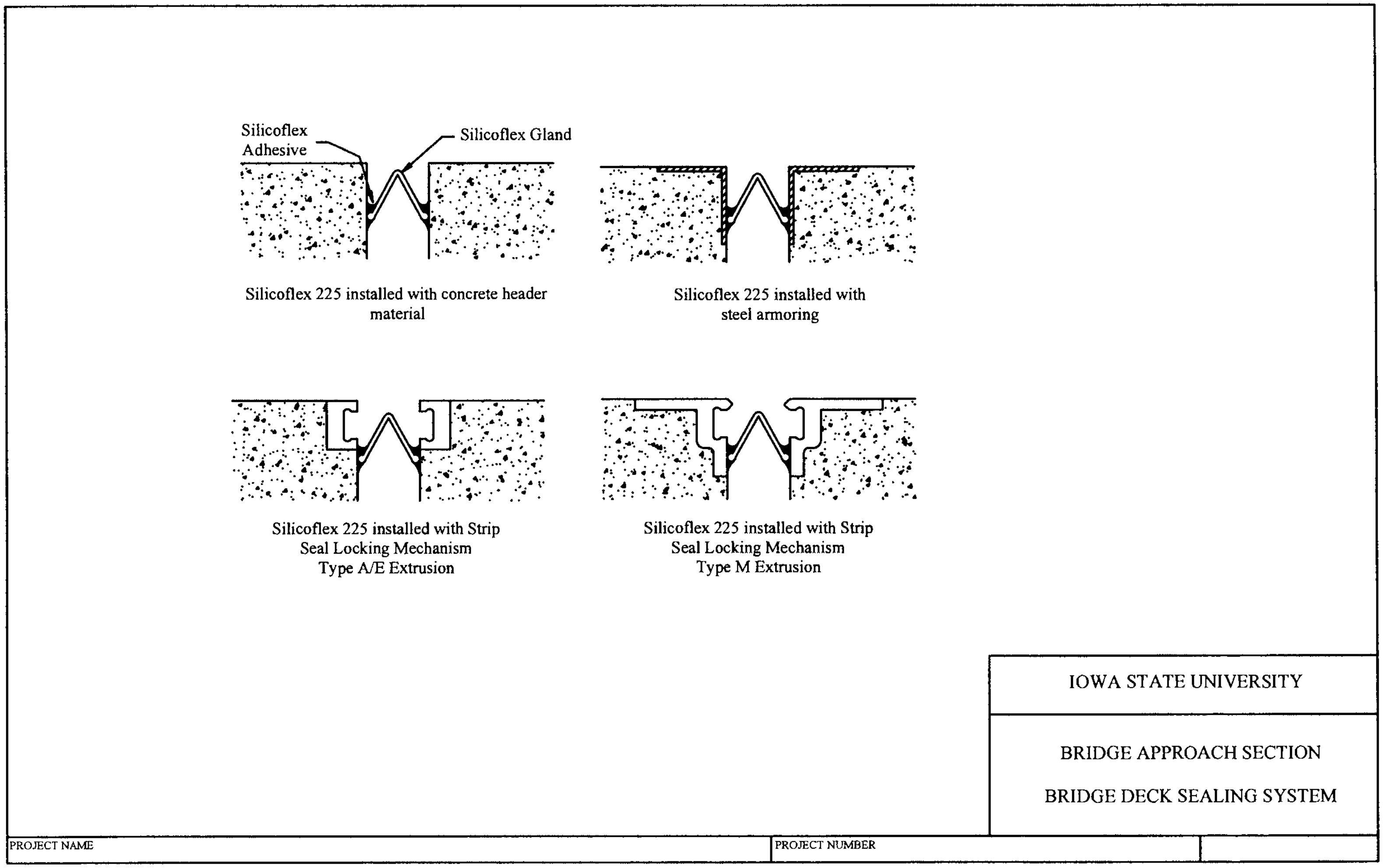


Figure 5.1 - V-shaped Rubber Gland Joint Sealing System

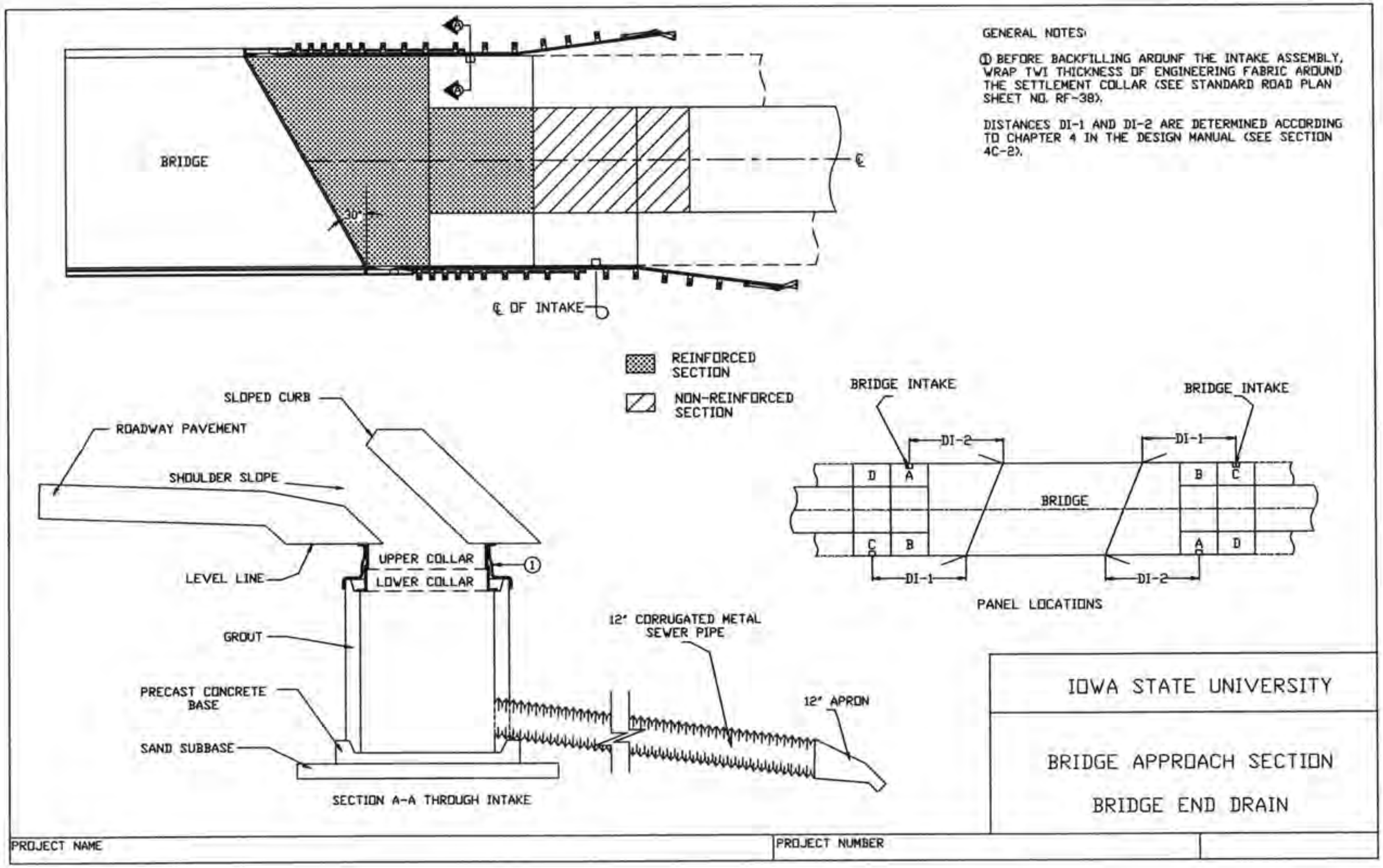
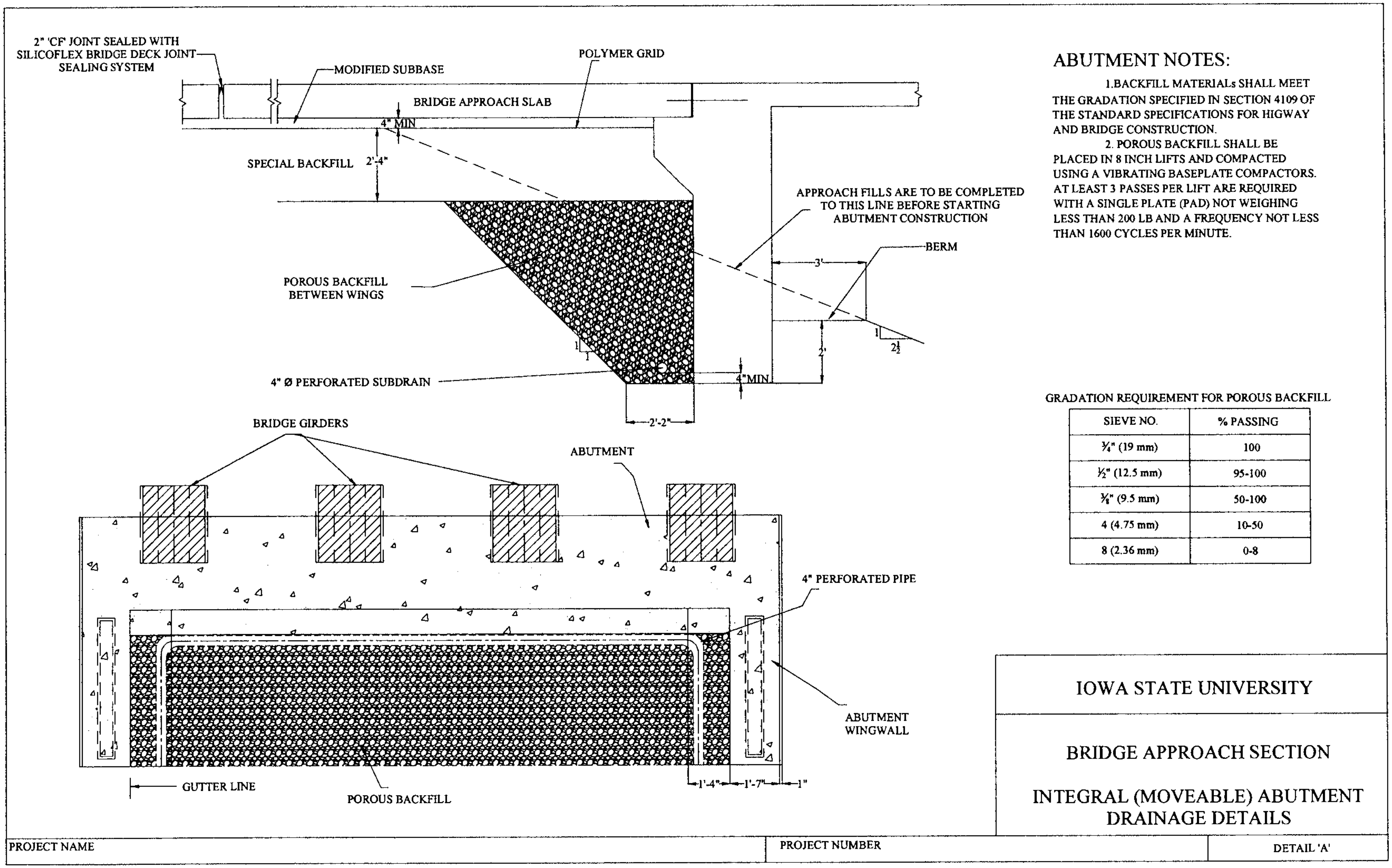


Figure 5.2 - Recommended Bridge End Drain Detail



SIEVE NO.	% PASSING
¾" (19 mm)	100
½" (12.5 mm)	95-100
⅜" (9.5 mm)	50-100
4 (4.75 mm)	10-50
8 (2.36 mm)	0-8

Figure 5.3 - Drainage Detail 1; using Porous Backfill behind the Bridge Abutment

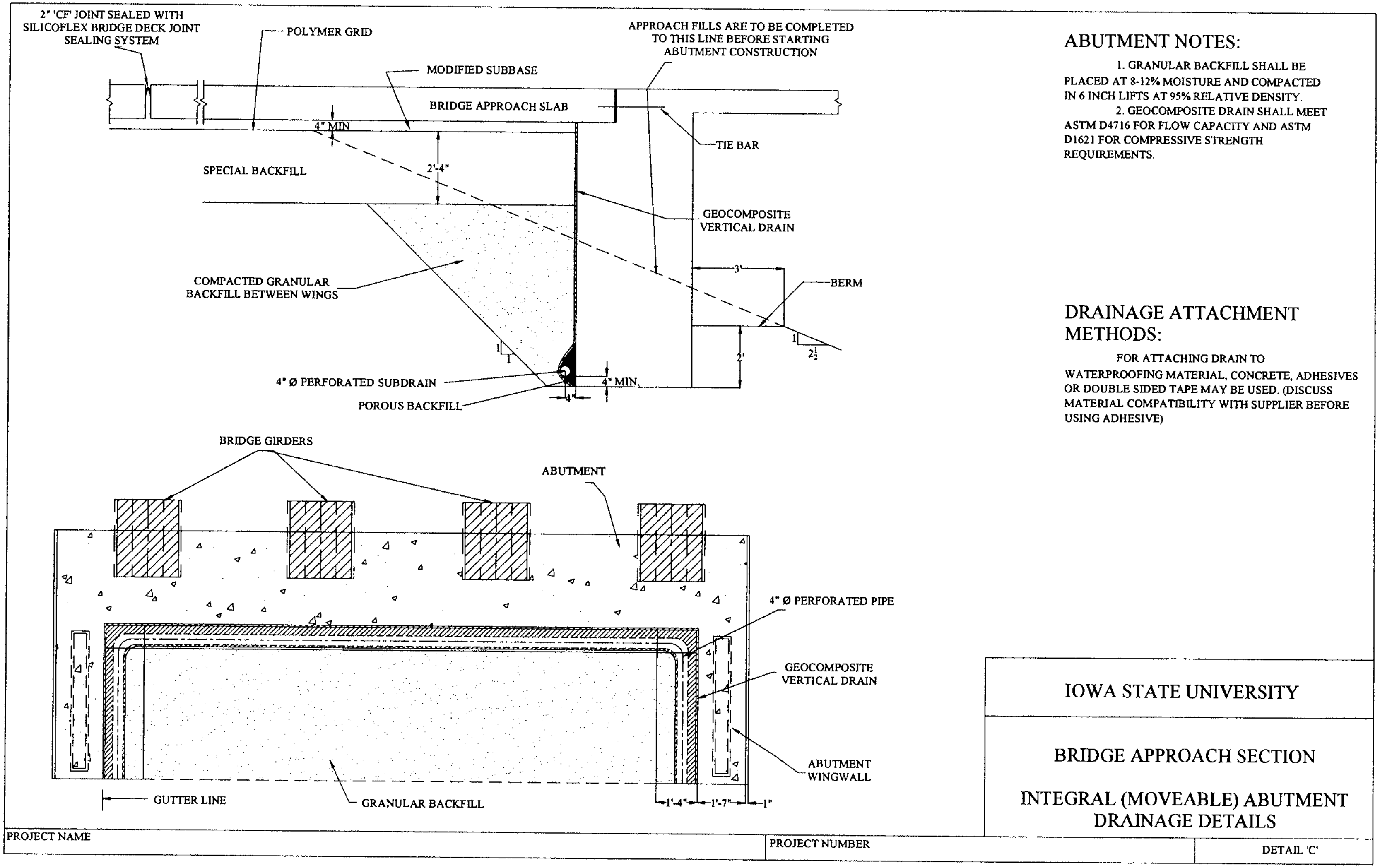


Figure 5.4 - Drainage Detail 2; using Geocomposite Vertical Drain

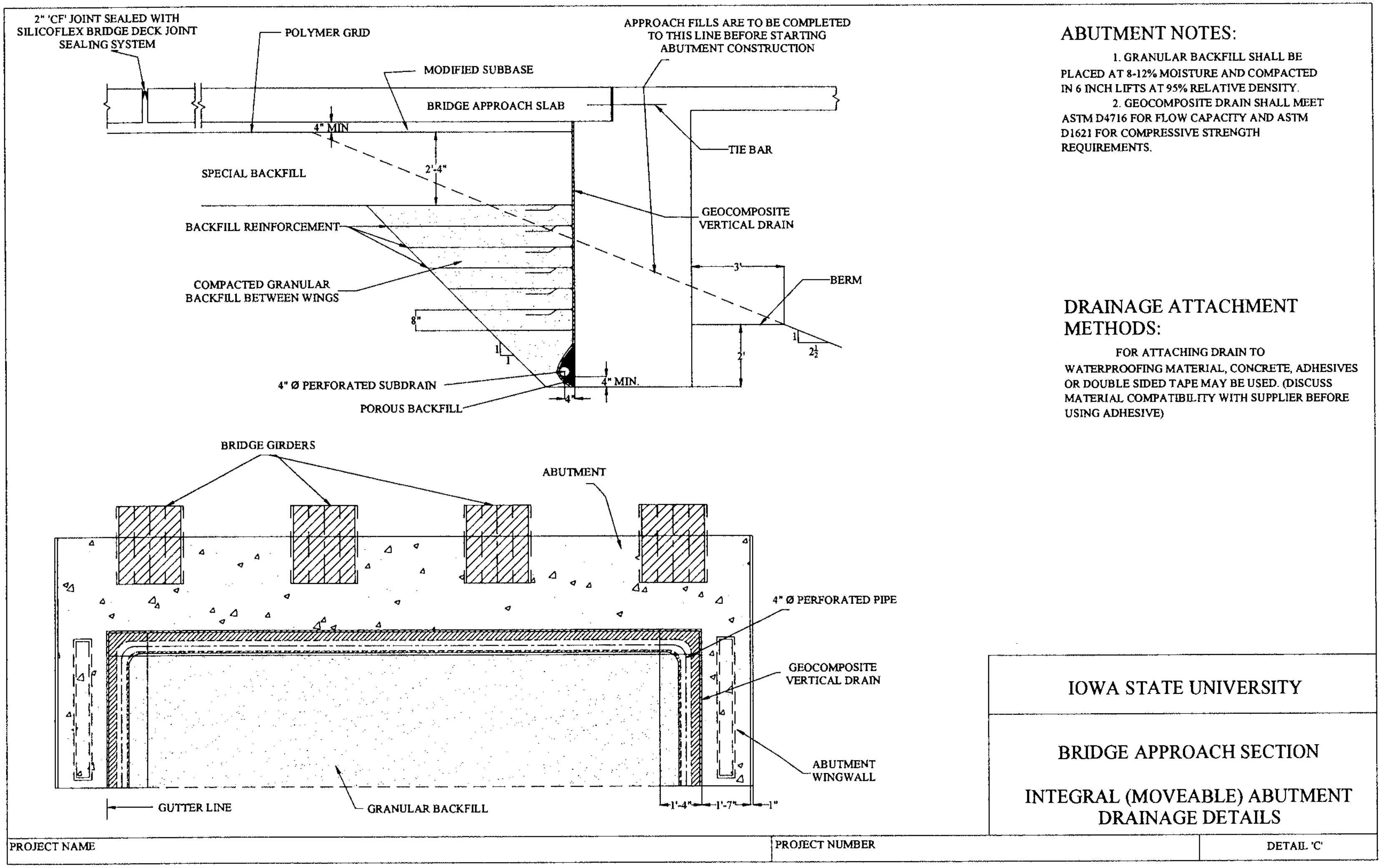
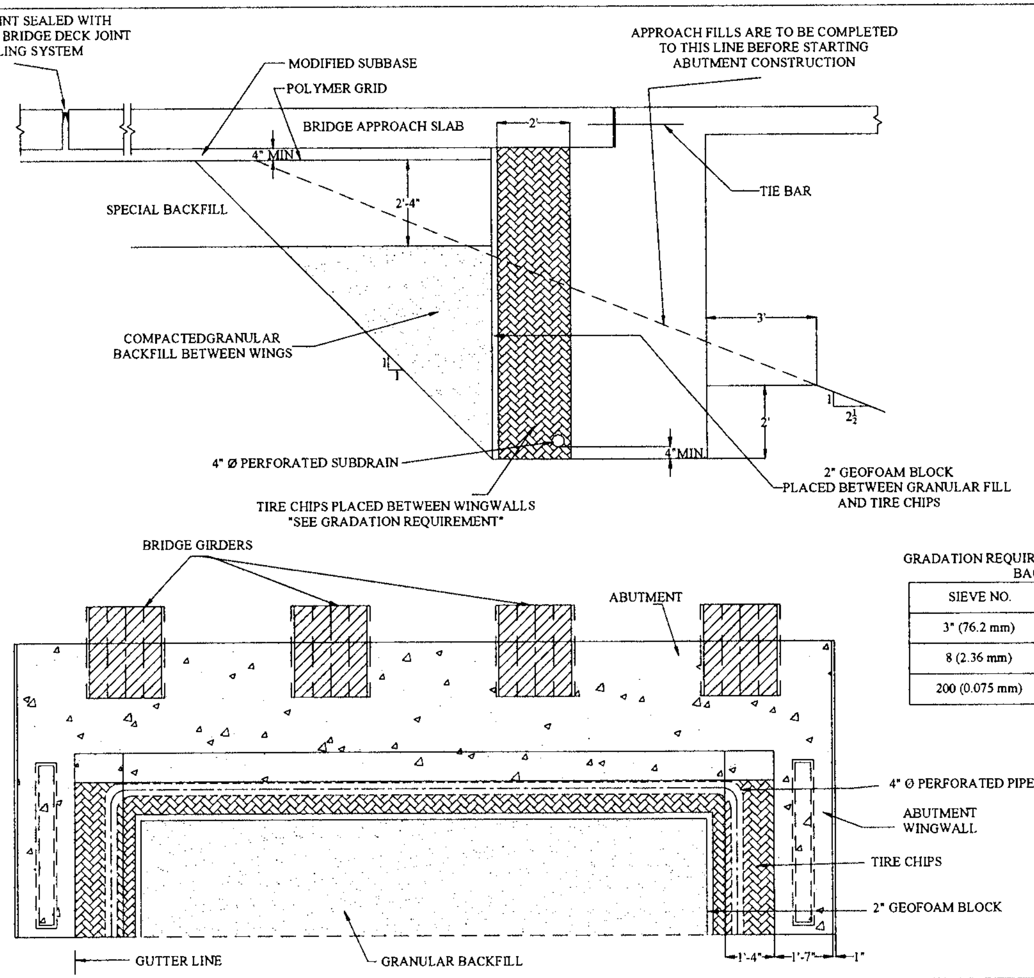


Figure 5.5 - Drainage Detail 3; using Geocomposite Vertical Drain with Backfill Reinforcement



ABUTMENT NOTES:

1. NONWOVEN GEOTEXTILES USED AS BACKFILL REINFORCEMENT SHALL MEET THE REQUIREMENT FOR A CLASS 1 PERMANENT EROSION CONTROL GEOTEXTILE AND STABILIZATION GEOTEXTILE ACCORDING TO AASHTO M288-96.
2. BACKFILL MATERIAL SHALL MEET THE GRADATION SPECIFIED IN SECTION 4109 OF THE STANDARD SPECIFICATIONS FOR HIGWAY AND BRIDGE CONSTRUCTION.
3. GRANULAR BACKFILL SHALL BE PLACED AT 8-12% MOISTURE. THE BOTTOM LAYER SHALL BE 10 INCH THICK WHILE THE SUBSEQUENT LAYERS SHALL BE 6 INCHES THICK. ALL LIFTS SHALL BE COMPACTED TO 95% RELATIVE DENSITY.

GRADATION REQUIREMENT FOR GRANULAR BACKFILL

SIEVE NO.	% PASSING
3" (76.2 mm)	100
8 (2.36 mm)	20-60
200 (0.075 mm)	50-100

GRADATION REQUIREMENT FOR TIRE CHIPS

SIEVE NO.	% PASSING
3/4	100
3/8	100-95
4	70-50
200	0-2

IOWA STATE UNIVERSITY

BRIDGE APPROACH SECTION

INTEGRAL (MOVEABLE) ABUTMENT DRAINAGE DETAILS

PROJECT NAME | PROJECT NUMBER | DETAIL 'D'

Figure 5.6 - Drainage Detail 4; using Tire Chips behind the Bridge Abutment

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APPENDIX A - WATER MANAGEMENT BRIDGE APPROACH MODEL

Table A1 - CONTECH C-60NW Nonwoven Geotextile Specifications

		Minimum average roll values	
Property	**Test Method**	**English**	**Metric**
Physical			
Weight	ASTM D4533	5.0 oz/sy	170 g/m^2
Thickness	ASTM D5199	60 mils	1.5 mm
Mechanical			
Grab Tensile Strength	ASTM D4632	160 lbs	712 N
Grab Elongation	ASTM D4632	50 %	50 %
Puncture Strength	ASTM D4833	85 lbs	378 N
Mullen Burst	ASTM D3786	280 psi	1930 kPa
Trapezoidal Tear	ASTM D4533	60 lbs	267 N
Hydraulic			
Apparent Opening Size (AOS)	ASTM D4751	70 US Std Sieve	0.212 mm
Permittivity	ASTM D4491	1.30 sec^{-1}	1.30 sec^{-1}
Permeability	ASTM D4491	0.24 cm/sec	0.24 cm/sec
Water Flow Rate	ASTM D4491	110 gpm/ft^2	4482 l/min/m^2
Endurance			
UV Resistance	ASTM D4355	70 %	70 %
(% retained after 500 hours)			

Table A2 - CONTECH C-80NW Nonwoven Geotextile Specifications

		Minimum average roll values	
Property	**Test Method**	**English**	**Metric**
Physical			
Weight	ASTM D4533	6.5 oz/sy	220 g/m^2
Thickness	ASTM D5199	70 mils	1.778 mm
Mechanical			
Grab Tensile Strength	ASTM D4632	205 lbs	912 N
Grab Elongation	ASTM D4632	50 %	50 %
Puncture Strength	ASTM D4833	110 lbs	490 N
Mullen Burst	ASTM D3786	350 psi	2413 kPa
Trapezoidal Tear	ASTM D4533	85 lbs	378 N
Hydraulic			
Apparent Opening Size (AOS)	ASTM D4751	80 US Std Sieve	0.180 mm
Permittivity	ASTM D4491	1.50 sec^{-1}	1.50 sec^{-1}
Permeability	ASTM D4491	0.38 cm/sec	0.38 cm/sec
Water Flow Rate	ASTM D4491	110 gpm/ft^2	4482 l/min/m^2
Endurance			
UV Resistance	ASTM D4355	70%	70%
(% retained after 500 hours)			

Table A3 - Structural Geogrid BX1100 Specification

Product Properties			
Index Properties	**Units**	**MD Values**	**XMD Values**
Aperture dimensions	mm (in)	25 (1.0)	33 (1.3)
Minimum rib thickness	mm (in)	0.76 (0.03)	0.76 (0.03)
Load Capacity			
True initial modulus in use	kN/m (lb/ft)	250 (17,140)	400 (27,420)
True tensile strength at 2% strain	kN/m (lb/ft)	4.1 (280)	6.6 (450)
True tensile strength at 5% strain	kN/m (lb/ft)	8.5 (580)	13.4 (920)
Structural Integrity			
Junction efficiency	%	93	
Flexural stiffness	mg-cm	250,000	
Aperture Stability	kg-cm/deg	3.2	
Durability			
Resistance to installation damage	%SC/ %SW/ %GP	90/83/70	
Resistance to long term degradation	%	100	

Table A4 - Tenax Ultra-Vera™ Geotextile Specifications

Property	Test Method	Units	Value
Resin			
Density	ASTM D1505	g/cm^3	0.94
Melt Flow index	ASTM D1238		1.0
Geocomposite			
Hydraulic properties			
Flow rate	ASTM D4716	lpm/mm	10.8
Coefficient of permeability		m/day	25,000
Reinforcement properties			
Tensile strength	ASTM D4595	lb/ft (kN/m)	2500 (36.5)
Number of load cycles before crack propagates			3000
Geonet core			
Thickness	ASTM D5199	mils (mm)	300 (7.6)
Creep reduction factor	GRI-GC8	-	1.14
Carbon black content	ASTM D4218	%	2.0-3.0
Nonwoven geotextile			
U.V. Resistance	ASTM G 154	%	95
Color			Orange
Serviceability class	AASHTO M-288		Class 1
Grab tensile	ASTM D4632	lbs (N)	202 (900)
Tear strength	ASTM 4533	lbs (N)	79 (350)
Puncture resistance	ASTM 4833	lbs (N)	79 (350)
CBR puncture resistance	ASTM 6241	lbs (N)	449 (2000)
AOS	ASTM 4761	US Std. Sieve (mm)	80 (0.18)
Permittivity	ASTM 4491	Sec^{-1}	0.5

Table A5 - STRIPDRAIN 75 Specifications

Property	STRIPDRAIN 75	Test Method
Core:		
Composition	High density polyethylene	
Thickness	0.75 in.	ASTM D5199
Compressive strength @ maximum 10% deflection	5,760 psf	ASTM D1621
Flow capacity @ 10 psi, i = 1.0	12 gal./min./ft. width (minimum)	ASTM D4716
Fungus resistance	No growth	ASTM G21
Moisture absorption	<0.05 %	ASTM D570
Geotextile (minimum average roll values):		
Grab tensile strength	95 lbs.	ASTM D4632
Grab elongation	50 %	ASTM D4632
Trapezoidal tear	40 lbs.	ASTM D4533
Mullen burst	180 lbs.	ASTM D3786
Puncture	45 lbs.	ASTM D4833
A.O.S.	70-100	ASTM D4751
Water flow rate	170 gal./min. per sq. foot.	ASTM D4491
Coefficient of permeability	0.20 cm/sec	ASTM D4491
Standard roll dimensions:		
Width	12", 18", 24", 30", and 36"	
Length	180'	

APPENDIX B - IRI GRAPHS

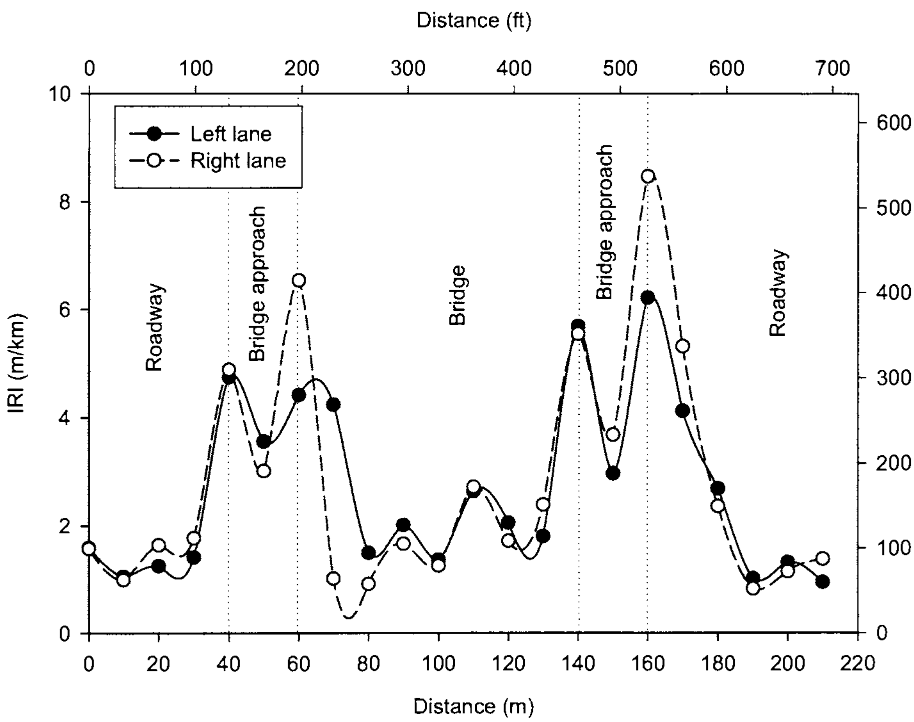

Figure B1 - IRI graph (Bridge No. 7773.0065 - SBL)

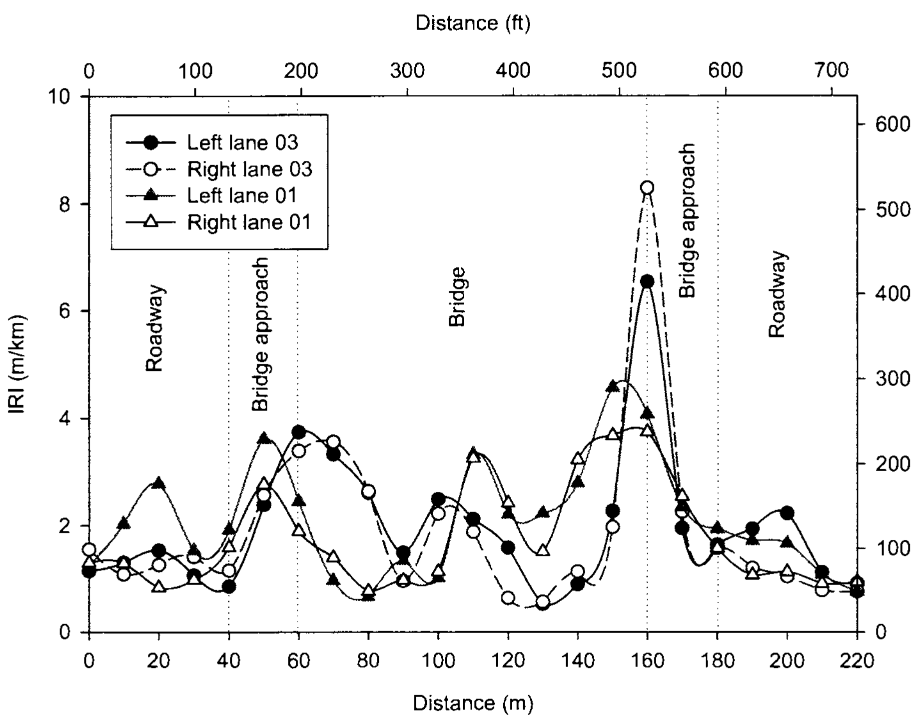

Figure B2 - IRI graph (Bridge No. 7774.L065 - SBL)

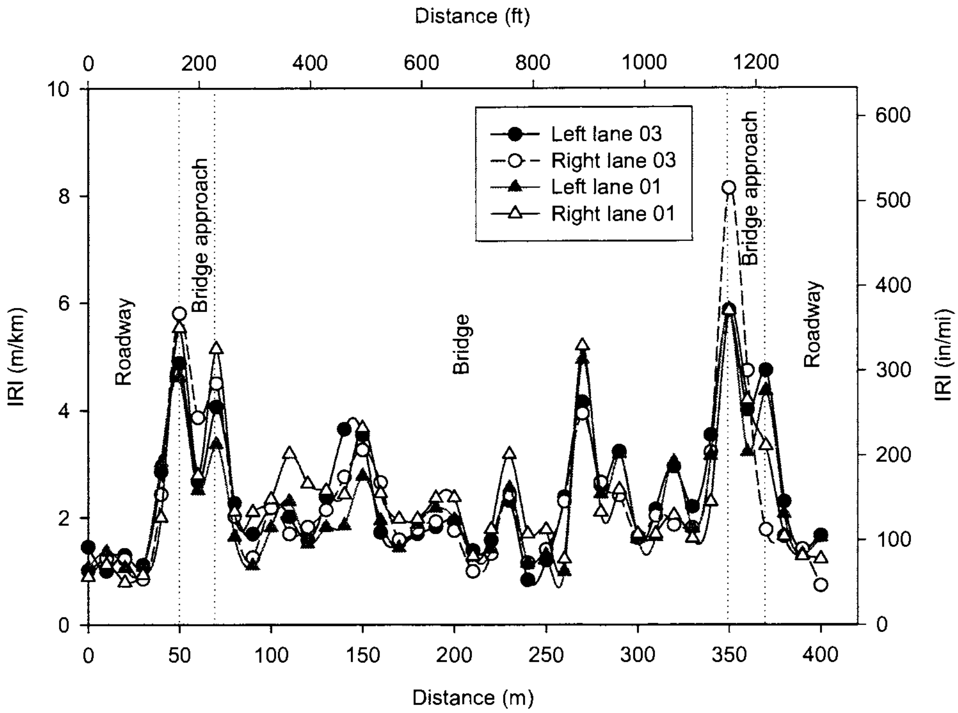

Figure B3 - IRI graph (Bridge No. 7776.8065 - NBL)

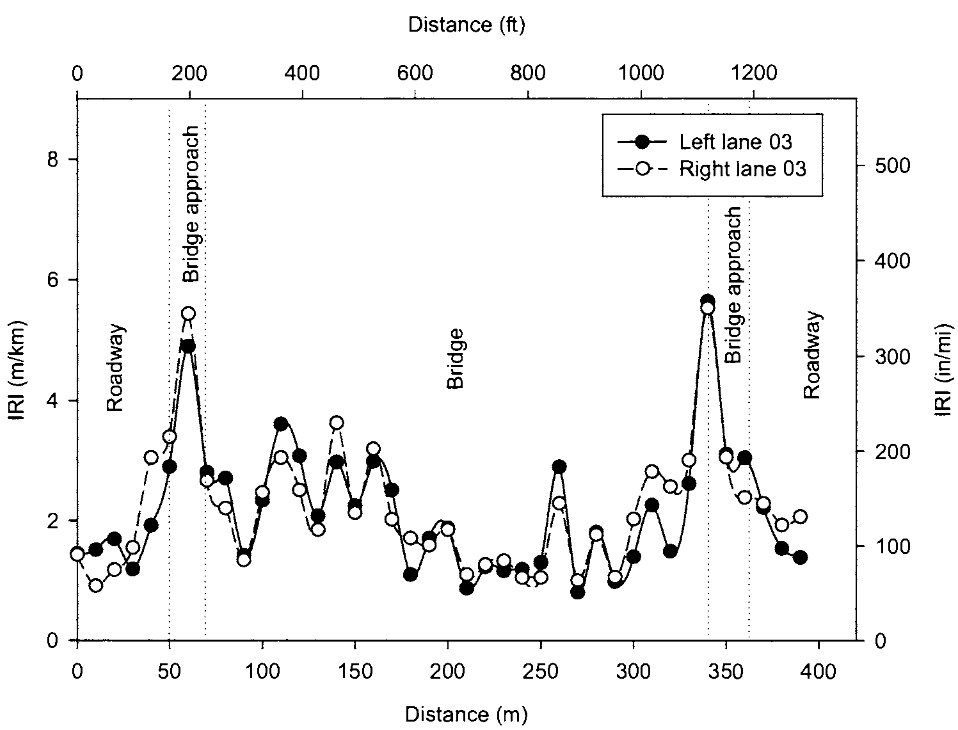

Figure B4 - IRI graph (Bridge No. 7776.8065 - SBL)

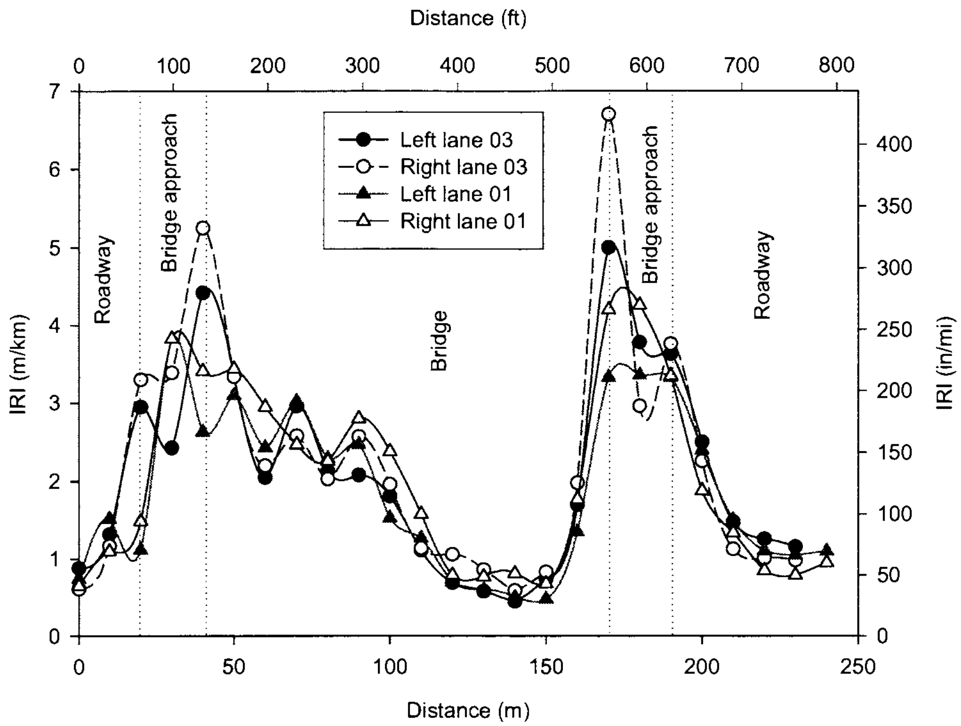

Figure B5 - IRI graph (Bridge No. 7777.0065 - NBL)

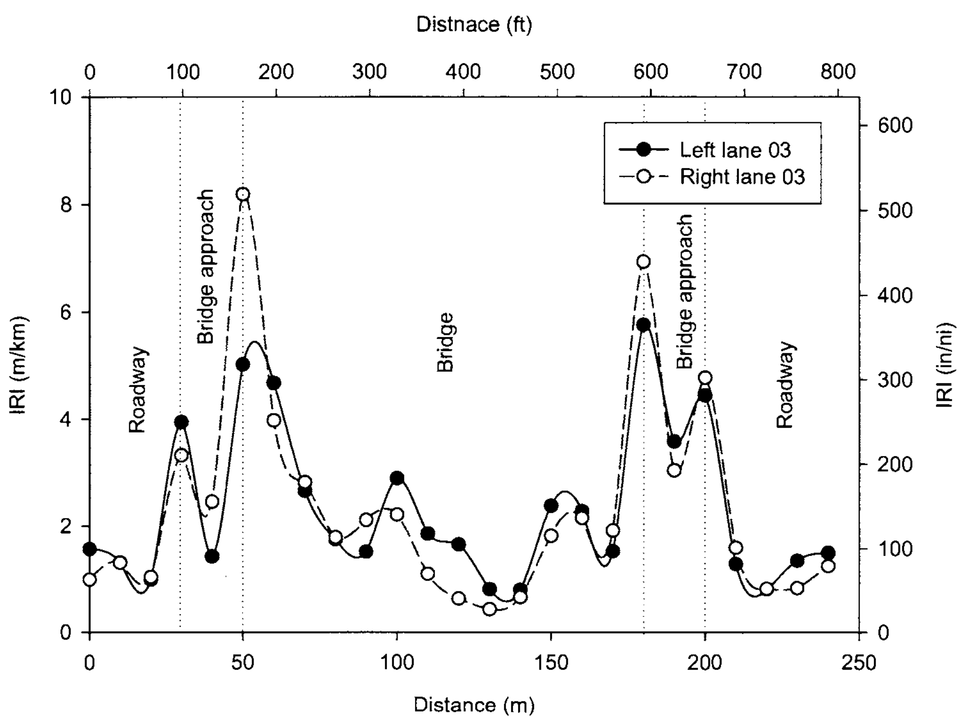

Figure B6 - IRI graph (Bridge No. 7777.0065 - SBL)

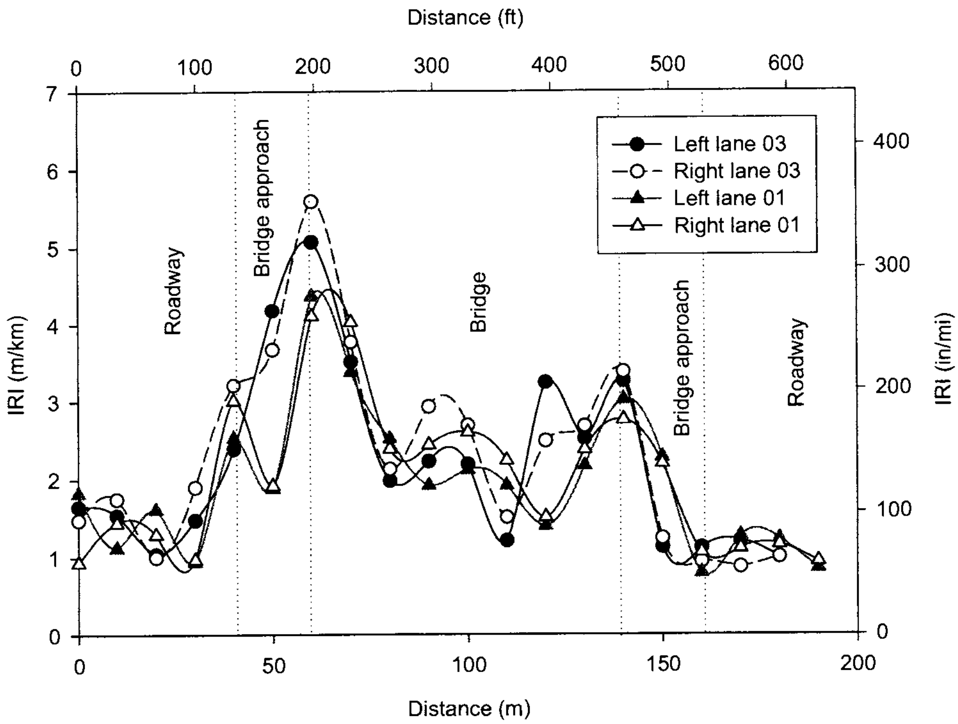

Figure B7 - IRI Graph (Bridge No. 7777.3065 - SBL)

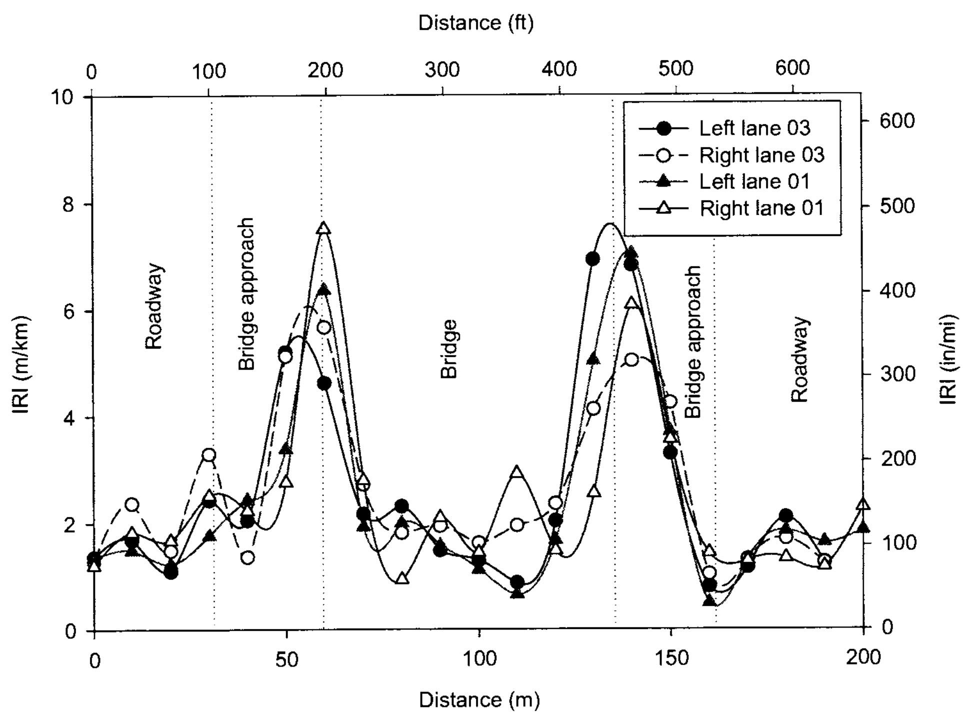

Figure B8 - IRI Graph (Bridge No. 7778.1065 - NBL)

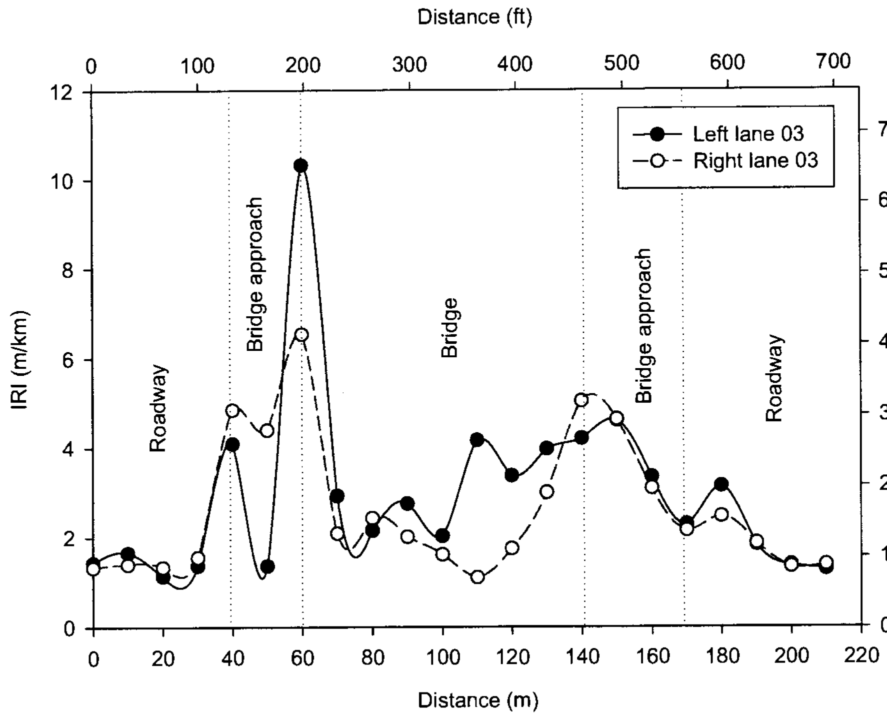

Figure B9 - IRI Graph (Bridge No. 7778.1065 - SBL)

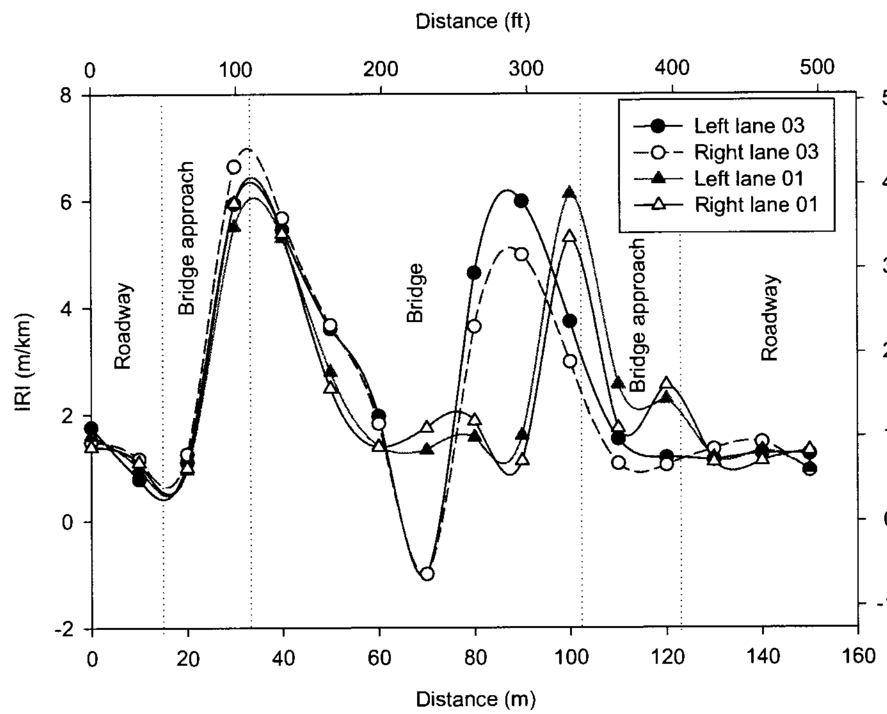

Figure B10 - IRI Graph (Bridge No. 7779.0065 - NBL)

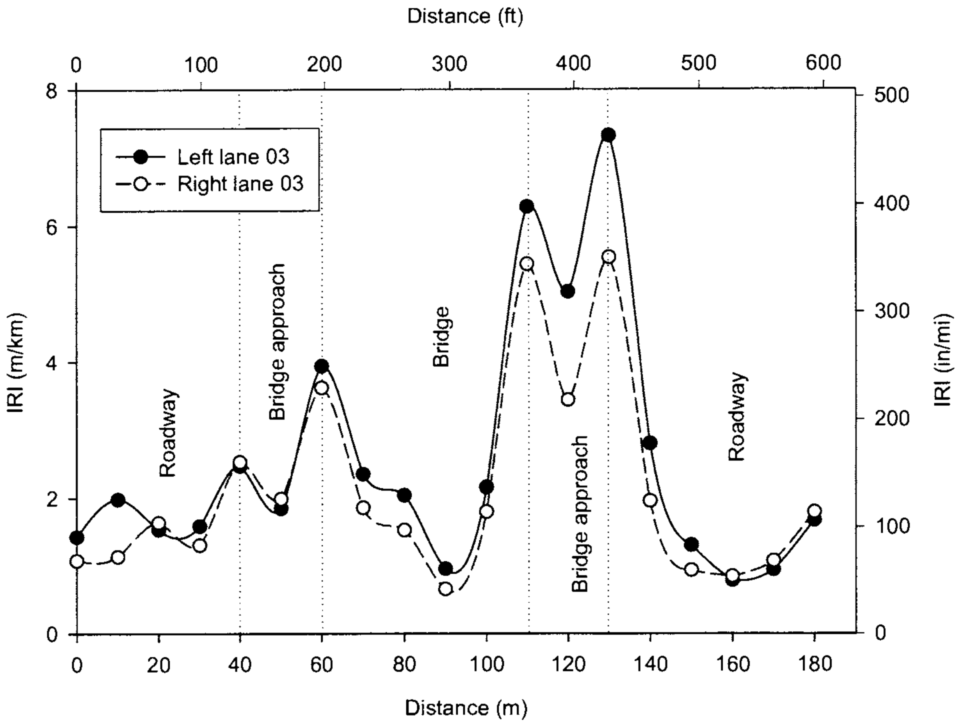

Figure B11 - IRI Graph (Bridge No. 7779.0065 - SBL)

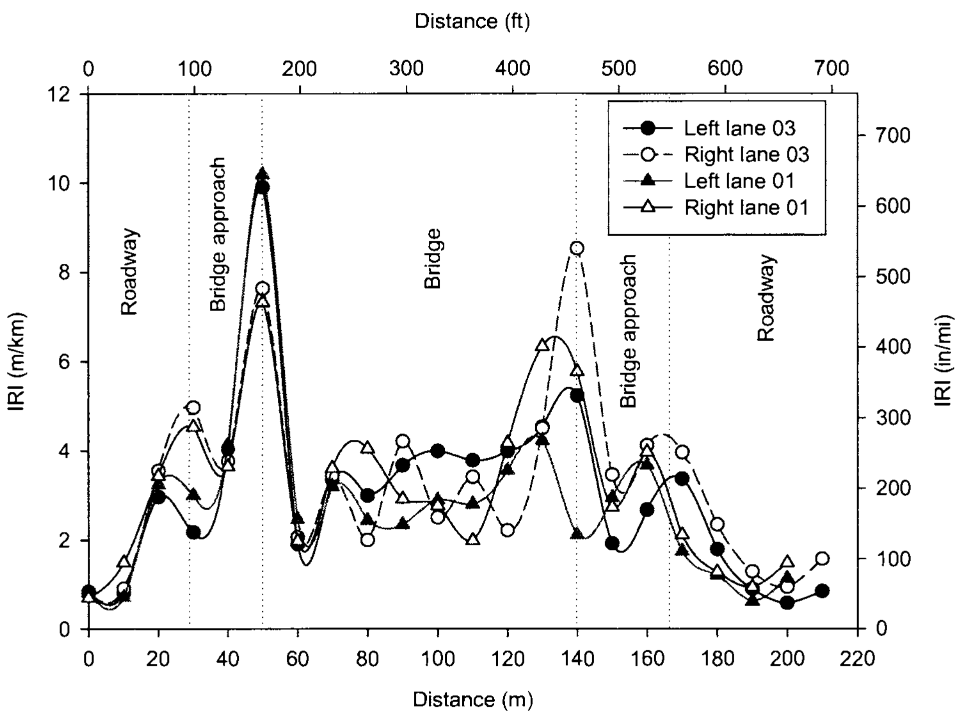

Figure B12 - IRI Graph (Bridge No. 7779.4065 - NBL)

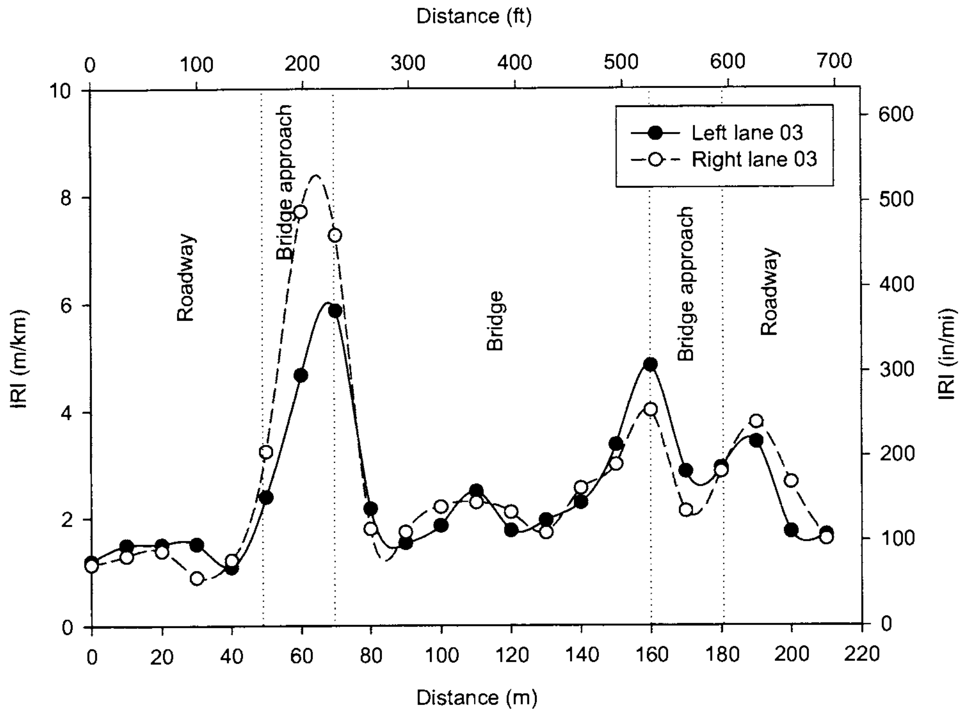

Figure B13 - IRI Graph (Bridge No. 7779.4065 - SBL)

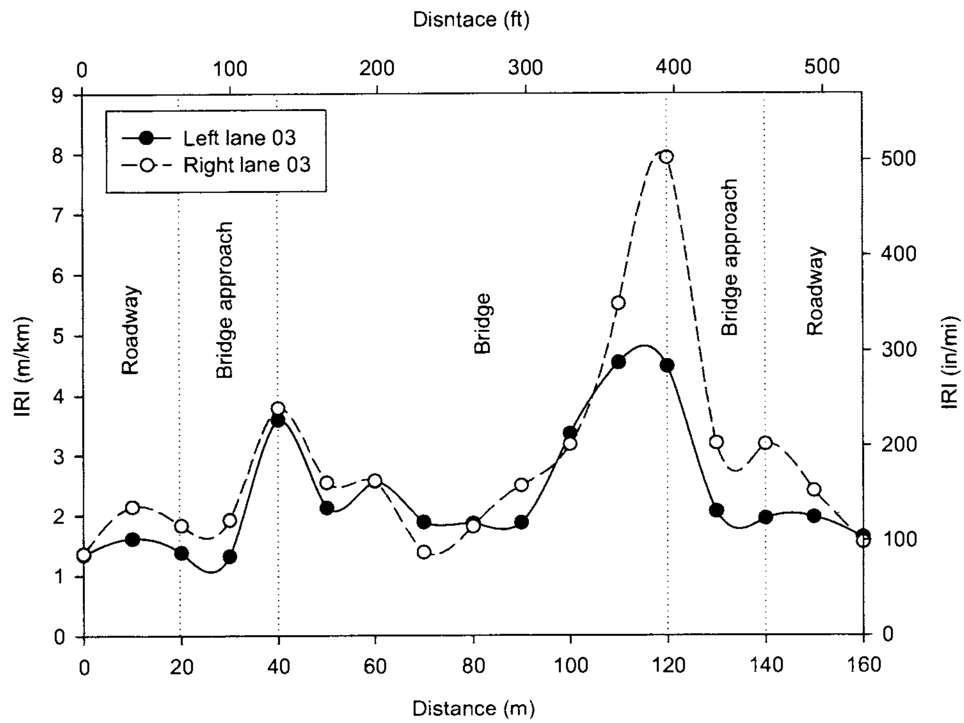

Figure B14 - IRI Graph (Bridge No. 7780.8065 - NBL)

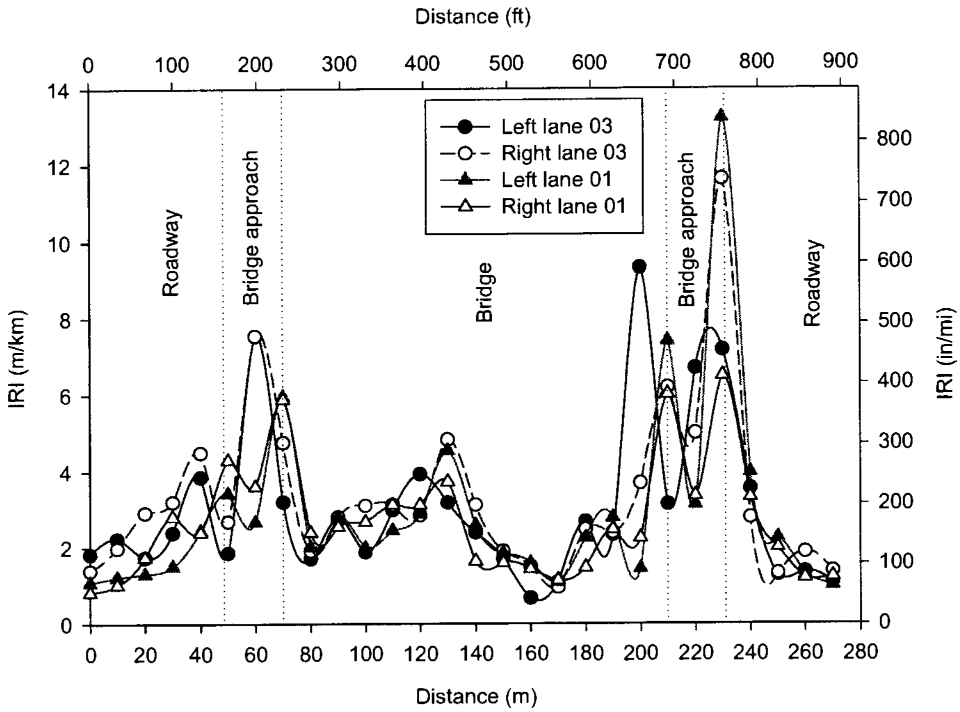

Figure B15 - IRI Graph (Bridge No. 7781.2065 - NBL)

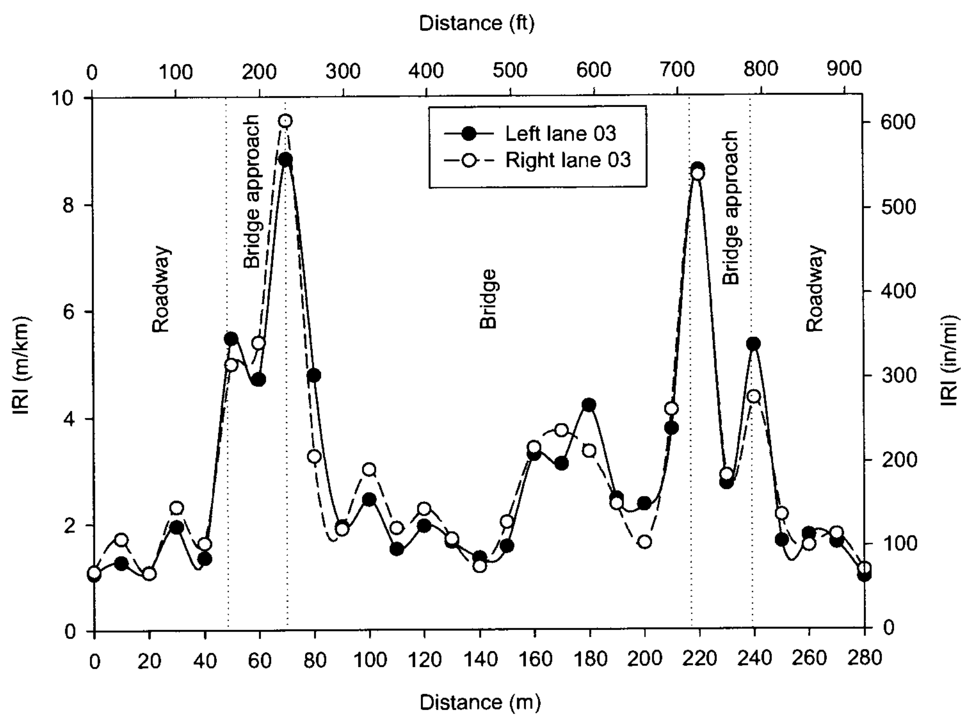

Figure B16 - IRI Graph (Bridge No. 7781.2065 - SBL)

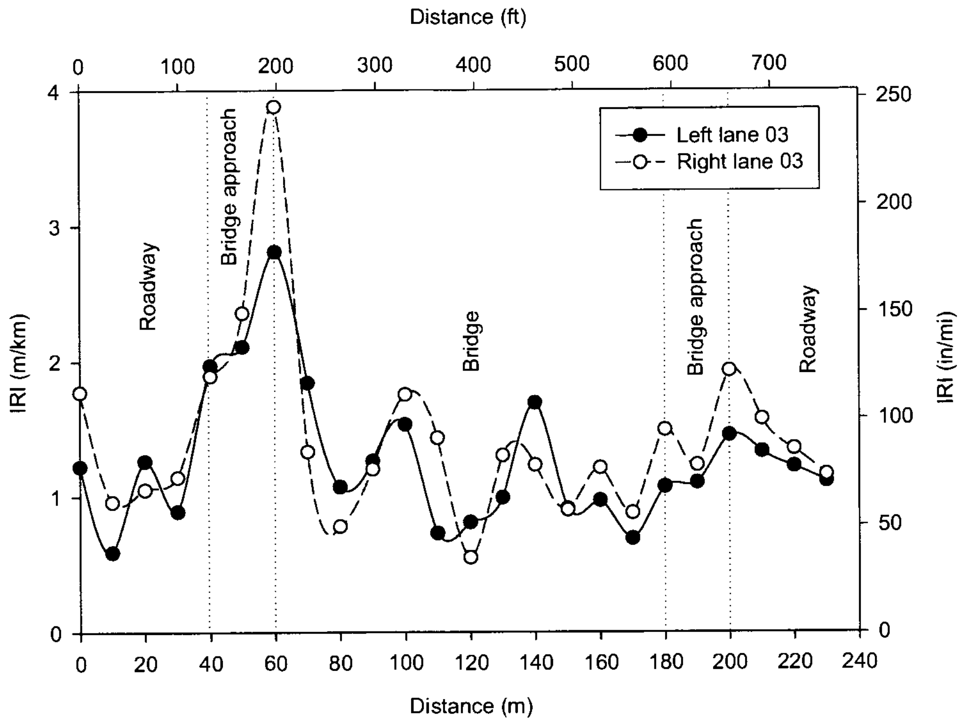

Figure B17 - IRI Graph (Bridge No. 7782.8L065 - SBL)

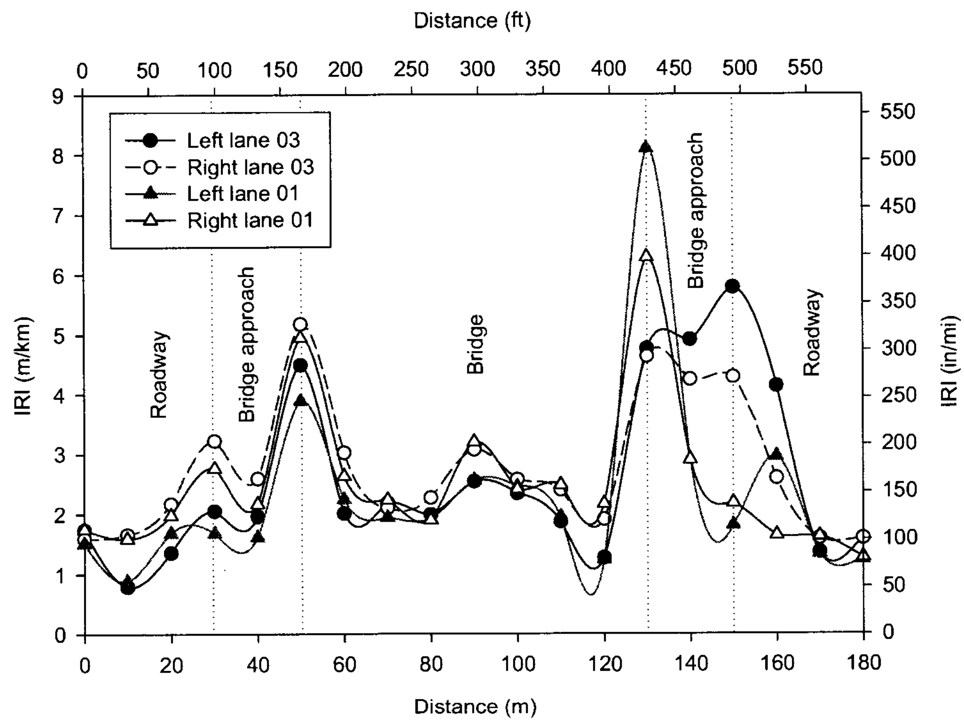

Figure B18 - IRI Graph (Bridge No. 7783.1065 - NBL)

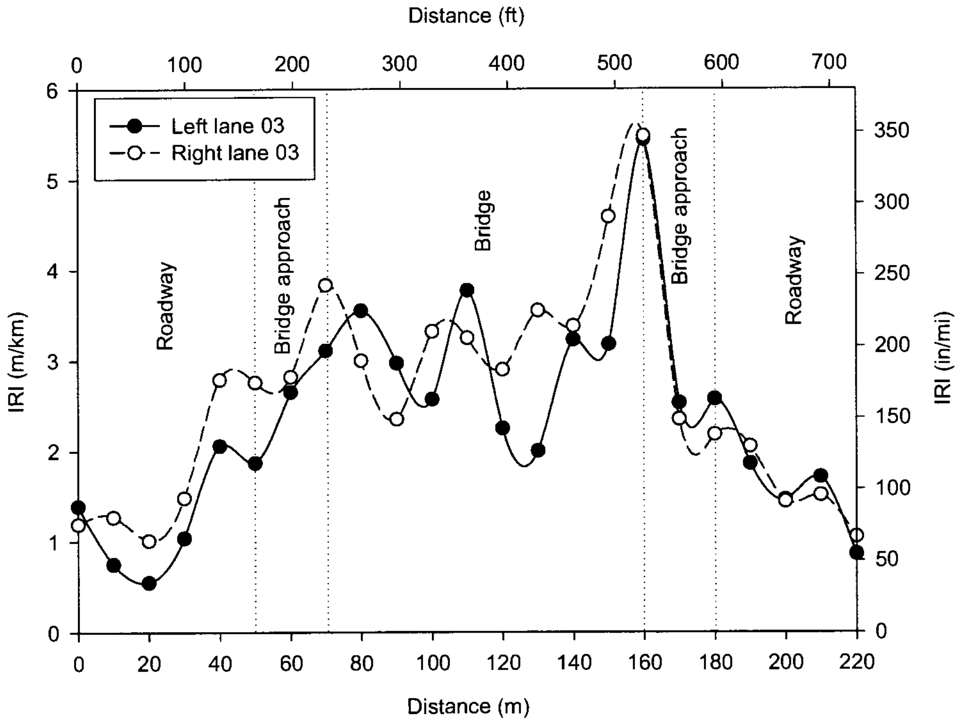

Figure B19 - IRI Graph (Bridge No. 7783.1065 - SBL)

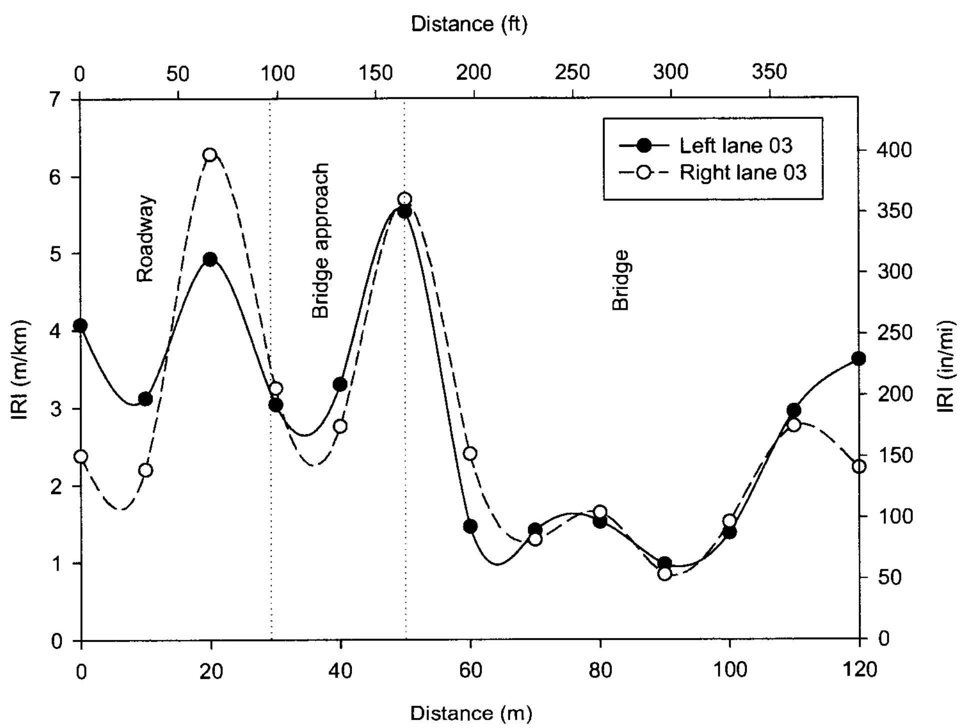

Figure B20 - IRI Graph (Bridge No. 9193.2R005 - NBL)