APPLIED APPROACH SLAB SETTLEMENT RESEARCH, DESIGN/CONSTRUCTION

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Approach embankment settlement is a pervasive problem in Oklahoma and many other states. The bump and/or abrupt slope change poses a danger to traffic and can cause increased dynamic loads on the bridge. Frequent and costly maintenance may be needed or extensive repair and reconstruction may be required in extreme cases. Research critically investigated the design and construction methods in Oklahoma to reveal causes and solutions to the bridge approach settlement problem. The major objectives of the research were: 1) Investigate causes of, and solutions to the approach slab settlement problem in available literature. 2) By direct investigation, determine primary causes of approach slab settlement for selected bridges in Oklahoma. Bridge configurations studied included those commonly used by ODOT and representing different embankment and foundation soil conditions typically encountered in Oklahoma. 3) Recommend solutions to minimize or eliminate approach slab settlement problems associated with Oklahoma bridges. 4) Recommend construction solutions to minimize potential for approach settlement problems. The major tasks completed included: 1) A review of available published literature related to the bridge approach settlement problem. 2) A survey of ODOT Field Divisions to solicit information about bridge sites experiencing settlement problems. In addition, potential sites were identified through discussions with key persons in the ODOT Materials Division. 3) Of the potential test sites, field reconnaissance investigations were conducted for 30 bridges at 22 separate locations in Oklahoma. These sites were identified as having moderate to severe problems and were representative of different bridge types, different geology, and different ages. To the extent possible, design, construction and maintenance records were obtained for these bridges. 4) At five of the test sites, subsurface investigation was conducted including: drilling and sampling, cone penetrometer testing, laboratory classification testing and oedometer testing to determine settlement parameters. 5) Statistically analyzed data to determine if there were relationships observed between bridge / embankment / foundation features and observed distresses. 6) Analyzed settlement of foundation soils and wetting-induced collapse settlement in embankment soils. 7) Developed recommendations for design and construction methods for addressing the approach slab settlement problem. The investigation revealed that erosion under the approach slab and under the abutment is a serious problem for many Oklahoma bridges. Consolidation of foundation soils was also found to be an important contributor to the approach slab settlement problem.</td></tr>
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(MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
in	Inches	25.4	millimeters	mm
ft	Feet	0.305	meters	m
yd	Yards	0.914	meters	m
mi	Miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	Acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	Gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	Ounces	28.35	grams	g
lb	Pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
mm	millimeters	0.039	inches	in
m	Meters	3.28	feet	ft
m	Meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	Liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	Grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF
ILLUMINATION				
lx	Lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SUMMARY

Approach embankment settlement is a pervasive problem in Oklahoma and many other states. The bump and/or abrupt slope change poses a danger to traffic and can cause increased dynamic loads on the bridge. Frequent and costly maintenance may be needed or extensive repair and reconstruction may be required in extreme cases. Research critically investigated the design and construction methods in Oklahoma to reveal causes and solutions to the bridge approach settlement problem. The major objectives of the research were: 1) Investigate causes of, and solutions to the approach slab settlement problem in available literature. 2) By direct investigation, determine primary causes of approach slab settlement for selected bridges in Oklahoma. Bridge configurations studied included those commonly used by ODOT and representing different embankment and foundation soil conditions typically encountered in Oklahoma. 3) Recommend solutions to minimize or eliminate approach slab settlement problems associated with Oklahoma bridges. 4) Recommend construction solutions to minimize potential for approach settlement problems. The major tasks completed included: 1) A review of available published literature related to the bridge approach settlement problem. 2) A survey of ODOT Field Divisions to solicit information about bridge sites experiencing settlement problems. In addition, potential sites were identified through discussions with key persons in the ODOT Materials Division. 3) Of the potential test sites, field reconnaissance investigations were conducted for 30 bridges at 22 separate locations in Oklahoma. These sites were identified as having moderate to severe problems and were representative of different bridge types, different geology, and different ages. To the extent possible, design, construction and maintenance records were obtained for these bridges. 4) At five of the test sites, subsurface investigation was conducted including: drilling and sampling, cone penetrometer testing, laboratory classification testing and oedometer testing to determine settlement parameters. 5) Statistically analyzed data to determine if there were relationships observed between bridge / embankment / foundation features and observed distresses. 6) Analyzed settlement of foundation soils and wetting-induced collapse settlement in embankment soils. 7) Developed recommendations for design and construction methods for addressing the approach slab settlement problem. The investigation revealed that erosion under the approach slab and under the abutment is a serious problem for many Oklahoma bridges. Consolidation of foundation soils was also found to be an important contributor to the approach slab settlement problem.

Chapter 1 INTRODUCTION

1.1 PROBLEM STATEMENT

Approach embankment settlement is a pervasive problem in Oklahoma and many other states. Quite often, the result of this settlement manifests itself in the form of damage to the approach slab leading up to the bridge and/or abrupt displacements between transitions from pavement to slab or slab to bridge depending on the design. The bump and/or abrupt slope change poses a danger to traffic and can cause increased dynamic loads on the bridge. Thus, frequent and costly maintenance is needed or sometimes extensive repair and reconstruction may be required in extreme cases.

1.2 OVERVIEW OF PROBLEM AND RESEARCH

In older bridges it was not uncommon to see approach slabs without a mechanical connection to the bridge, which resulted in an abrupt displacement (bump) at the approach slab-bridge interface (joint) as depicted in Figure 1.1.

Figure 1.1 Displacement at Bridge Approach Slab-Bridge Interface due to Differential Settlement

This displacement is largely related to the fact that abutments are pile supported and experience little settlement while the approach slab rests on abutment backfill and embankment soil that in turn is supported by the underlying foundation soil. Thus, the problem is really a differential settlement problem resulting from compression of the soil strata below the approach slab. It is now relatively common for the approach slab to rest on the edge of the abutment and be mechanically linked (via reinforcing steel) to the bridge slab. When settlement of the embankment and backfill occurs, rather than an abrupt displacement at the joint, the unsupported end of the approach slab at the pavement end moves downward while the end supported by the bridge rotates, as shown schematically in Figure 1.2. Settlement at one end results in an overall rotation of the slab about a relatively fixed position on the abutment. In this case the abrupt displacement at the joint is minimized; however, if the settlement below the approach slab is significant a loss of support (void) may occur under the slab resulting in a structural failure (severe cracking or breaking) of the slab. Furthermore, while the abrupt displacement at the end of the bridge is minimized, the rapid change of roadway slope due to rotation of an approach slab may be unacceptable for safe flow of traffic and results in increased dynamic loads on the bridge from truck traffic. In either scenario, i.e. abutment-supported or unsupported approach slab, significant maintenance and repair are often required.

While differential settlement due to the compression of underlying soil strata is an obvious problem, there are many other factors that may cause or

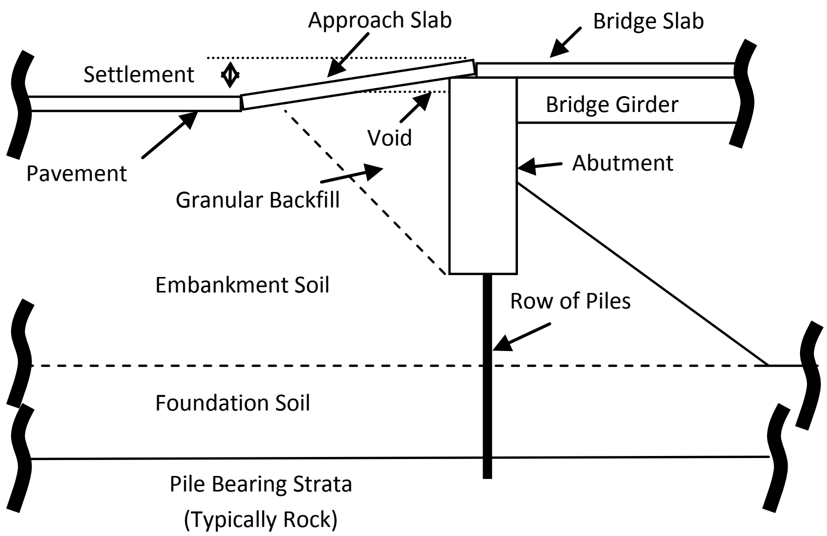

Figure 1.2 Schematic Cut-Away Cross-Section View of Possible Consequence of Approach Slab Settlement Accompanied by Void Formation, Based on Integral Abutment Bridge (Not to Scale)

amplify the problem. Some other factors that may contribute to approach slab settlement include but are not limited to erosion of supporting soil (pavement layers and underlying fill), compaction of material immediately below the slab due to cyclic traffic loading, and lateral deformation of wing walls and loss of confinement in the abutment backfill. These problems may be exacerbated by poor drainage beneath the approach slab and in the backfill.

Over approximately two years, the research has critically investigated the design and construction methods employed in Oklahoma to determine causes and solutions to the bridge approach settlement problem. This was accomplished through an extensive review of the literature to examine problems experienced and solutions employed by other state departments of transportation, a detailed forensic analysis of bridge approaches in Oklahoma that have experienced these problems, and critical analysis of current design and construction methods currently used in Oklahoma. Solutions that work in concert with current Oklahoma bridge design and construction practices are presented, which include suggested alterations to existing designs and modification of specifications and quality control methods to improve performance. In addition, more comprehensive and innovative approaches to designing the approach embankment-abutment-approach slab system are presented.

1.3 RESEARCH OBJECTIVES AND TASKS

The major objectives of the research were as follows:

1) Investigate causes of, and solutions to the approach slab settlement problem in available literature to determine their relevance to Oklahoma problems and gain insights into possible solution strategies.

2) By direct investigation, determine primary causes of approach slab settlement for selected bridges in Oklahoma. Bridge configurations studied included those commonly used by ODOT and representing different embankment and foundation soil conditions typically encountered in Oklahoma. This investigation included among other things, a critical review of the design, construction and post construction records related to each bridge, and a thorough field investigation at selected bridge sites.

3) Recommend solutions to minimize or eliminate approach slab settlement problems associated with Oklahoma bridges. This includes solutions related to the geometric configuration and structural details of bridge features such as abutments, wing walls, approach slab and foundation (typically piles); geotechnical details related to embankment geometry, soil type, and

compaction requirements; abutment backfill requirements related to material type, compaction methods, geometry and sub-drainage; and approach slab details including structural design, slab support, drainage layers, and connections at the interface between the bridge and roadway pavement. Solutions include modifications to existing designs as well as entirely new design strategies. In addition, recommendations for proper subsurface investigation and laboratory testing to adequately characterize critical soil behavior are given.

4) Recommend construction solutions to minimize potential for approach settlement problems. This includes recommendations for construction practices, inspection and quality control, and suggested modifications to existing specifications and development of new specifications as appropriate.

The major tasks completed to accomplish these objectives included:

1) Conducted a review of available published literature related to causes of and solutions to the bridge approach settlement problem.

2) Conducted a survey of ODOT Field Divisions to solicit information about bridge sites experiencing settlement problems. In addition, potential sites were identified through discussions with key persons in the ODOT Materials Division. Through this process, 49 potential tests sites representing all but two field divisions were identified for possible research.

3) Of the 49 potential test sites, field reconnaissance investigations were conducted for 30 bridges at 22 separate locations in Oklahoma. These sites were identified as having moderate to severe problems and were representative

of different bridge types, different geology, and different ages. To the extent possible, design, construction and maintenance records were obtained for these bridges.

4) At five of the test sites, extensive subsurface investigation was conducted including: drilling and sampling, cone penetrometer testing, laboratory classification testing and oedometer testing to determine settlement parameters.

5) Statistically analyzed data collected from surveys, ODOT records, and field investigations to determine if there were relationships observed between bridge/embankment/foundation features and observed distresses.

6) For sites where oedometer data was obtained, used simple analytical procedures to examine possible settlement of foundation soils and wetting-induced volume change in embankment soils.

7) Developed recommendations for design and construction methods for addressing the approach slab settlement problem.

1.4 ORGANIZATION OF REPORT

Chapter 2 of this report presents a literature review of the approach settlement problem and potential solutions. Chapter 3 presents the research methodology with respect to the site reconnaissance visits and information collected, subsurface exploration methods employed in the field at selected sites and laboratory testing conducted on samples obtained. Chapter 4 discusses the results of the field reconnaissance visits with the emphasis on settlement observations and erosion observed. Chapter 5 presents results of the subsurface exploration, laboratory testing, settlement prediction and discussion associated with five test sites selected for more in-depth study. Finally, in Chapter 6 a summary and recommendations for addressing the bridge approach settlement problem are presented.

Chapter 2 LITERATURE REVIEW

2.1 INTRODUCTION

This chapter details literature relevant to this study. It begins with an overview of the bridge approach settlement issue and the effects of differential settlement between the bridge and the approach slab. Then the causes behind the settlement of approach slabs, such as foundation settlement, fill settlement, and erosion are covered. Finally, the results of previous studies on current and alternative designs and construction practices are presented.

2.2 EFFECTS OF BRIDGE APPROACH SETTLEMENT

2.2.1 Overview

The bridge approach settlement issue has been investigated previously. Settlement of the embankment and foundation soil, erosion, and poor construction practices were identified as the major causes leading to approach settlement (White et al. 2007a). Differential settlement results in a “bump” at the interface between the bridge and the approach. The settlement of the bridge is typically much less than that of the adjacent roadway or embankment (Briaud 1997, Puppala et al. 2009). Bakeer et al. (2005) claim that primary and secondary settlement of the embankment fill and natural foundation soils are the most significant factors contributing to approach slab settlement.

Bridge approach settlement is a complex problem; the causes and solutions are site-dependent. Therefore, there is no single fix for the issue (Briaud et al. 1997). Proposed solutions include reinforcing foundation soil and/or the backfill, improving drainage systems, using filter wraps to prevent erosion, and structurally designing the approach slab (White et al. 2007a).

2.2.2 Extent and Costs

According to a synthesis study by Briaud et al. (1997), about 25% of bridges in the United States are affected by the bump at the end of the bridge. The bump can damage vehicles, impact traffic velocity and create unsafe driving conditions (White et al. 2005, Lin et al. 1999).

It is estimated that maintenance costs due to the bump at the end of the bridge are at least $100 million annually (Briaud et al. 1997). A variety of maintenance approaches are used: overlaying the approach with asphalt to smooth the transition, filling the void with grout, or injecting polyurethane which expands to lift the slab up. These techniques will temporarily smooth the transition, but do not address the long term causes at the site. The “bump” also lowers the public image of transportation agencies (White et al. 2005).

2.3 CAUSES OF BRIDGE APPROACH SETTLEMENT

2.3.1 Settlement of Foundation Materials

There have been numerous investigations on the relationship between approach slab movement and settlement of the foundation soil. Wahls (1990) claims that the behavior of the foundation soil is one of the most important factors in the performance of a bridge approach. Immediate settlement of the foundation soil occurs during the application of the load and is not generally a problem; primary consolidation and secondary consolidation are time dependent and are significant factors regarding bridge approach settlement.

Therefore, cohesive foundation soils present a more significant problem than non-cohesive soil regarding approach settlement (Puppala et al. 2009).

Addressing primary and secondary consolidation of the foundation soil is imperative to alleviating approach settlement issues (Wahls 1990). Soft clays have a low shear strength, high compressibility, and low permeability. These characteristics cause large vertical and lateral displacements as well as excess pore-water pressures (Lin et al. 1999). Without ground improvement, embankments over thick soft clay deposits can have large settlement issues (Miao et al. 2009). For example, a highway bridge over the River Tees in England was built on a soft alluvial clay deposit. The western approach, where consolidation was addressed by adding surcharge prior to construction, had no differential settlement issues. The eastern approach, where consolidation was not addressed, had significant settlement with the approach (Jones et al. 2008).

2.3.2 Compression of Fill Materials

2.3.2.1 Wetting-Induced Collapse

Wetting-induced collapse settlement occurs when the post-construction moisture content is increased by events such as precipitation, capillary water from the foundation soils, or flooding. Four factors that have a significant influence in wetting-induced collapse problems are the soil type, total overburden stress, pre-wetting moisture content, and dry unit weight (e.g. Lim and Miller 2004). The densification can occur over a few months or years for partially saturated fills; the time required depends chiefly on the occurrence and rate of water infiltration. The pre-wetting conditions and potential post-compaction changes in density, moisture and stress are important in predicting collapse potential. The moisture changes are typically more significant near exposed surfaces (Lawton et al. 1992). Lim and Miller (2004) performed oedometer tests on 22 Oklahoma soils. They found that collapse occurred over a wide range of dry unit weight and that non-cohesive sandy soil exhibited negligible collapse potential while clayey soils showed significant collapse potential. Their research found that even “slight” collapse potential can still lead to significant settlement as embankment height increases. They recommended that specifications should have more stringent compaction requirements and increased quality control, particularly for high embankments.

The drier a soil is during compaction, the greater the potential for collapse. Soil dry of optimum tends to have significant matric suction which provides meta-stable bonding of aggregated soil particles. Upon wetting, the suction is reduced, which can cause the soil structure to experience volumetric compression if the overburden pressure is significant. Increasing the relative compaction substantially reduces the wetting-induced collapse, increases the critical overburden pressure at which collapse is maximized, and increases wetting-induced swelling potential. Therefore, a balance must be sought between collapse and swell when specifying compaction (Lawton et al. 1989). Improper compaction control can result in approach settlement and other problems.

As mentioned, quality control is critical in achieving high quality compacted fills, which is necessary to combat fill settlement. In Kansas, research showed that current practices allowed for highly variable compaction of the finished subgrade and that quality control devices typically assess less than one percent of the compacted area (Rahman et al. 2007).

2.3.2.2 Nondurable Material Causing Settlement

Durable rocks do not slake in the presence of water; however, nondurable rocks will slake when subjected to water and compressive forces. When this happens, the nondurable rocks break down into soil and fill voids in the soil structure. This leads to settlement of the embankment. Laboratory testing is recommended to predict the settlement effects from slaking nondurable rocks. For embankments made of sedimentary rocks, the soaked compression test, developed by Strohm, has been used (Vallejo and Pappas, 2010).

2.3.3 Erosion

White et al. (2007a) conducted a study of 74 bridge sites, consisting of bridges that were performing well and others that were performing poorly. In their study, they determined that severe erosion of the backfill was a critical issue in the settlement of approach slabs. Severe erosion was observed in 40% of the investigated bridges, of which 14 were integral and 16 were non-integral. This number includes erosion under the approach pavement, under the bridge abutment, and at the sides of the abutment. Erosion can expose the abutment piles and cause the loss of backfill under the approach.

2.3.3.1 Suffusion of Granular Materials

Suffusion is a process where water seeping through a granular soil dislodges fine particles without destroying the coarse grain soil matrix. Such soils are called “internally unstable.” Suffusion can occur when soil has large and small particles such that the large particles form a skeleton that the smaller particles can pass through. When water flows it can transport the loose particles away through the voids. The fines in these matrices are in the voids and may not support effective stress (Indraratna et al. 2011, Shwiyhat and Xiao 2010). Soils with high porosity and those that are gap-graded tend to be more susceptible to suffusion (Wan and Fell 2008). In laboratory investigations, suffusion erosion has been observed to begin at hydraulic gradients much less than the critical gradient (Ahlinhan et al. 2010, Wan and Fell 2008). While suffusion is less catastrophic than piping, it can lead to increased permeability, seepage, and consolidation of a soil layer (Wan and Fell 2008, Shwiyhat and Xiao 2010).

2.3.3.2 Piping

Piping is the interaction of fluids and solids where flowing water creates a drag force that carries soil particles (Liang et al. 2011). Piping erosion occurs when erosion creates channels in the soil that resemble pipes. The soil is transported through these pipes to an exterior face and away from the embankment. Sandy and silty soils are the most susceptible (Hagerty 1991). The cavities formed in this process can become large and collapse (Sinco et al. 2010).

Adams and Xiao (2011) proposed mixing organic soil with sand to increase the sand’s resistance to piping. In their investigation, they studied the use of organic soil to reduce susceptibility to piping, and whether the organic mixes would be suitable for use as a backfill material. The investigation used sandy-soil from a construction stockpile and commercially available “compost” consisting of an equal proportion of green-waste and bio-solids. Their investigation found a positive correlation between organic content in the soil and the reduction of erosion. The use of bioremediation resulted in a large increase in compressibility of the backfill material. Therefore, bioremediation was found to be unsuitable for use as embankment backfill material.

2.3.3.3 Temperature Cycles

Seasonal temperature variation affects the differential settlement between approach and bridge, especially in integral bridges (Puppala et al. 2009). Thermal cycles are significant to integral abutment bridges because the abutment expands and contracts with the bridge deck. During expansion, the fill material is compacted. Then, a void is created when the bridge and abutment contract (Lowell et al. 2008). This void increases erosion potential (Puppala et al. 2009). Minnesota reports that seasonal expansion pushes the approach slab back, but friction between the slab and soil prevents it from staying with the bridge deck during contraction. The gap between the two surfaces fills with debris and widens with each cycle, allowing water into the fill beneath the slab (Lowell et al. 2008).

2.4 BRIDGE DESIGN DETAILS

2.4.1 Abutment

2.4.1.1 Integral Abutment

Integral abutments have a monolithic rigid connection between the abutment and bridge deck, and are allowed to move laterally with the bridge deck slab. The approach consists of the approach slab, the approach fill, the backfill, and the foundation soil (Puppala et al. 2009). In a survey, many state transportation agencies listed approach settlement as a major concern for integral abutments. Other problems reported include cracking of the abutment back wall, the deck, the approach slab, and wing wall (Maruri and Petro 2005).

2.4.1.2 Non-integral Abutment

Non-integral or conventional abutments support the bridge deck using roller and pin connections. Lateral movement of the bridge is allowed by expansion joints without transmitting load or movement to the abutment (Puppala et al. 2009).

2.4.2 Approach Slab Details

The rigidity of the approach slab plays a role in the severity of the bump. Reinforced approaches performed better than flexible approaches in one study (Puppala et al. 2009). In a study of various states' approach designs, states that use reinforcement bars to connect the bridge deck to the approach slab observed cracking in the approach slab, and the bump simply moving from the bridge-approach interface to the approach-pavement interface (Lowell et al. 2008).

Islam (2010) analyzed approach slab drawings from various state DOTs. Using finite element analysis and MathCAD, he determined that many approach slabs were under-designed. As the soil beneath the slab settles, the slab loses support and deflects, causing a bump. Cai et al. (2005) analyzed the approach slab structurally as a beam. This investigation also found that approach slabs are often under-designed, and the deflection resulting from loss of soil support leads to a bump. However, designing the slab as a simply supported beam with no soil support is too conservative in most cases. The investigation found that approach slabs are often built based on experience, as opposed to designed with engineering calculations. This often leads to inadequate approach slab designs and the resulting bump.

2.4.3 Drainage

Good bridge drainage is comprised of two systems: surface and subsurface. Surface drainage should take water from the roadway away from the embankment. Subsurface drainage should get excess water out of the embankment (Mekkawy et al. 2005). Improper drainage has been cited by several researchers and state departments of transportation as one of the most important factors in bridge approach settlement. Poorly functioning or non-functioning drainage systems lead to erosion and void development (Puppala et al. 2009). The Alabama Department of Transportation cited improper drainage as a major factor in the deterioration of its bridges (Ramey and Wright 1997).

Mekkawy et al. (2005) used physical models to observe the performance of the Iowa drainage design and several alternatives regarding settlement, void development, and flow rate. The typical Iowa design, consisting of porous backfill surrounding a perforated pipe, performed poorly; high settlements and low flow rates were observed. An alternative design, comprised of a vertical geocomposite placed between geotextile reinforced backfill and a sub-drain, performed very well; no void or settlement was observed and a high flow rate was achieved.

2.5 REINFORCEMENT OF THE FOUNDATION SOIL

2.5.1 Pile Supported Embankment

The use of timber or pre-cast concrete driven piles in the foundation soil beneath an embankment is expected to reinforce the soil and transfer the embankment loads to stiffer layers lying below the soft soil (Puppala et al. 2009). Using piles to support embankments over soft ground has demonstrated advantages. In case studies, Chen et al. (2010) found that using piles and geotextiles helped transfer embankment load to better layers without loading soft soil, and that the piles and embankment settled less than the foundation soil. Bakeer et al. (2005) conducted a study of a large number of pile-supported approach slabs in southeastern Louisiana. Using the Louisiana Department of Transportation and Development rating system for approach slabs, they found that pile-supported slabs typically had acceptable ratings. Seven representative bridges were selected for an in-depth investigation. It was concluded that there was a wide range of performance of pile-supported slabs in Louisiana, and the inconsistencies were largely due to differences in negative skin friction from site to site. Piles used in conjunction with wick drains and surcharging were effective in reducing settlement of bridge approach embankments over soft soil in New South Wales (Hsi 2007).

2.5.2 Deep Soil Mixing

Deep Soil Mixing (DSM) is an in situ soil treatment method in which soil is blended with cementitious materials, commonly referred to as “binders.” These binders are injected through hollow, rotated mixing shafts tipped with a cutting tool (Bruce 2000). The process improves soft natural soil by creating in-place soil-cement columns (Archeewa et al. 2011). Column diameters typically range from 0.6 to 1.5 m (Bruce 2000). Binders include cement, lime, fly ash, or combinations (Archeewa et al. 2011). For use as in situ reinforcement under embankments, DSM is usually made up of relatively closely-spaced single columns. In the DSM methods, it is important that a thorough and uniform mixing of the binder be achieved (Bruce 2000). An important consideration when using the DSM technique is the area ratio (ar). This is defined as the ratio between the treated area and the total unit area of soil, which is the summation of both treated and untreated plan area (Archeewa et al. 2011).

Compared to other methods for in situ soil reinforcement, deep mixing methods are economical for large projects in soft, compressible soils. The depth of DSM is limited to approximately 40 m; a large working space is needed and the method is not applicable to soils that are very dense, stiff, or may have boulders (Bruce 2000). The quality of the soil-cement columns depends on many factors, including the cement used, soil type and construction technology, and therefore may be inconsistent (Miao et al. 2009).

Puppala et al. (2007) studied the use of DSM in Texas in expansive soil deposits. They found the DSM technique to reduce soil movement. Also, the DSM technique was cost effective compared to other methods that were considered.

Saride et al. (2010a) presented a case study of DSM technology used to mitigate bridge approach settlement in a highway embankment constructed on soft lean clay in Arlington, Texas in late summer 2008. In this investigation, the south approach was constructed using DSM columns, while the north approach was constructed using conventional methods and considered as a control for the investigation. Saride et al. (2010a) found that the compressibility of the foundation soil was reduced about 3 times in this case. At the time of the study, the vertical deformation of both the embankment and foundation were within permissible limits, which were defined as 25 mm; however, the embankment was not fully open to traffic at the time of the study. Archeewa et al. (2011) investigated this case and created a numerical model to compare model predictions to actual results. They used Plaxis finite element software to model a section of embankment. The long-term settlement predictions matched well with the measured results. The model did not account for construction related settlement. The numerical model demonstrated that DSM systems have the potential to reduce settlement occurring at bridge approaches.

2.5.3 Controlled Modulus Columns

Controlled modulus columns (CMC) are constructed using a hollow auger which is drilled into the soil to a specific depth. A cement-based grout is pumped through the auger at a low pressure to form a column (Miao et al. 2009). The procedure has a dual effect of densification and reinforcement. The displacement auger is powered by equipment with a high torque and downward thrust capability and displaces the soil laterally without removing soil. A workable grout is injected at a low pressure, usually less than 145 psi. There is no soil mixing during grouting. The columns are typically installed in a square grid with spacing being between 3.9 and 9.8 feet. A typical column's diameter ranges between 14 to 20 inches (Plomteux et al. 2004, Pearlman and Porbaha 2006).

Miao et al. (2009) conducted an experiment on one cement grout mortar CMC in soft clay, as well as unimproved soil for comparison purposes. The study concluded that CMC can reduce settlement significantly. Pearlman and Porbaha (2006) present a case study of a CMC foundation system used for embankment support. The site consisted of a very soft alluvial clay layer with a thickness varying from 20 to 36 ft. Under this layer was a medium dense chalk layer with silt fragments and mixed with medium dense sand. They found that CMC systems provide acceptable performance and constructability. The authors noted that the uniform loading of the embankment will induce negative skin friction not accounted for in the tests, and may result in additional settlement.

2.5.4 Rammed Aggregate Piers

Rammed aggregate piers (RAP) are constructed by excavating a hole in the foundation soil and filling the hole with compacted aggregate (White et al. 2007b). The piers are installed in a grid to create a system of strong aggregate piers in a matrix of weaker soil (Thompson et al. 2009).

The use of rammed aggregate piers in a railway construction project in the Philippines was successful in two ways. First, the piers provided a drainage path for excess pore water, and primary consolidation was achieved in less than 5 weeks. Second, the total settlements were much less than what was estimated for an unreinforced embankment (Morales et al. 2011). No significant stiffness differences were observed between an isolated pier and an individual pier in a group in an investigation of group efficiency (White et al. 2007b). In a case study of RAPs supporting an MSE wall, the RAP reinforced system settled one sixth as much as the unreinforced soil (Thompson et al. 2009).

2.6 FILL MATERIALS AND CONSTRUCTION

2.6.1 Geotextile and Geogrid Reinforcement

The steep slopes present in embankment construction create the potential for lateral spreading, which can cause problems if it occurs adjacent to the abutment. Placing layers of geotextile into the embankment provides tensile support that can potentially reduce the lateral deformations (Edgar et al. 1987). According to Edgar et al. (1987), the use of geotextiles by the Wyoming Highway Department (WHD) has reduced the lateral and vertical deformation of the approach embankments and significantly reduced the differential settlement between approaches and abutments. When geotextiles are used to transfer loads to piles using the membrane effect, the stiffness of the geosynthetic directly affects the behavior (Hello and Villard 2009).

2.6.2 Geocell Mattress Foundation

A geocell mattress foundation is a 3D honeycomb series of interlocking cells that are constructed from polymer material. The cells confine the granular layer at the base of an embankment (Bush & Jenner 1990). Geocell systems can either be pre-manufactured, or fabricated at the site using geogrid material (Dash 2011). Geocell foundations are used to improve the stability and bearing capacity of soft soil. The uses of a geocell mattress foundation in a bridge embankment are to carry a flexible slab, to restrict the lateral deformation of a subgrade, distribute the embankment and vehicle loads more uniformly across the subgrade, and improve the tensile and shear strength of the approach subgrade (Jiang et al. 2011). Advantages of the geocell system are that it can immediately act as a working platform for embankment construction, it provides a rigid base to the embankment which leads to more uniform settlement, can minimize construction time, eliminate excavation and replacement costs, minimizes excessive settlements and lateral deformations (Latha et al. 2006).

Latha et al. (2006) performed a series of laboratory investigations on model embankments to investigate the performance of geocell foundations. A similar laboratory experiment was performed by Dash (2011). Latha et al. (2006) observed that in these experiments, geocell systems increased the surcharge capacity and decreased deformations compared to the unreinforced embankments. Uniaxial geogrid (UX) material demonstrated almost twice the surcharge capacity compared to the unreinforced model. Biaxial and non-oriented polymers performed about halfway in between these two extremes, with biaxial geogrid material performing slightly better than non-oriented polymers (Latha et al. 2006).

A geocell system was used to alleviate the bump at the end of the Yixi Bridge in Fushun, China. The settlement of the pavement was controlled mostly by the foundation settlement. The bump was not present in the approach using a flexible geocell system (Jiang et al. 2011).

2.6.3 Mechanically Stabilized Earth

MSE walls use geofabric or geogrid reinforcement wrapped around layers of compacted backfill to create a vertical, self-contained wall (Abu-Hejleh et al. 2008). The Colorado Department of Transportation (CDOT) has employed three alternatives to alleviate the bump at the end of the bridge, including flowable fill, mechanically stabilized earth (MSE) with well graded granular Class 1 backfill, and MSE with free-draining Class B filter material. However, the bump problem has persisted (Abu-Hejleh et al. 2008). CDOT then commissioned a study to improve the effectiveness of the procedure and to research means to make the procedure more economical. In an investigation using three MSE walls along two-lane highway embankments, settlement and piping were apparent immediately following construction and caused significant roadway damage. Therefore, preventing water infiltration is important in a MSE wall to avoid piping of the backfill material (Dodson 2010). A U-shaped MSE wall performed well in supporting a bridge abutment in a numerical analysis (Chang et al. 2006).

2.6.4 Lightweight Aggregate

Using a lightweight fill material reduces the embankment gravity load on the foundation soil. Examples of lightweight fill include lightweight aggregate, expanded polystyrene and lightweight concrete. Though the fill must be lightweight, it must still have high strength and stiffness and low compressibility (Puppala et al. 2009). Research was performed on the usage of light weight aggregate (LWA) material produced from expanded clays and shale (ECS) for its potential as fill material to reduce approach settlement. In a recent study by

Saride et al. (2010b), it was demonstrated that the material could reduce dead weight and resulted in high internal stability. The high internal friction angle reduced vertical and lateral forces. In compressibility and swell tests, the ECS was compared to typical fill materials and was found to have a lower bulk density by half, a higher friction angle, and a significantly lower compression index.

In addition to material tests, an embankment was constructed over 20-feet-thick soft moist clay layer underlain by a 10-feet-thick layer of dense sand underlain by hard sandstone. The Texas Department of Transportation used LWA material on the approach to reduce the load on the foundation soil on one end of the bridge, while the other end was constructed with normal embankment material. The ECS embankment performed satisfactorily regarding the bump at the end of the bridge. One issue that came up in the experiment was rotation due to localized bulging of the fill on the outside slope of the embankment induced by rainfall (Saride et al. 2010b).

2.6.5 Controlled Low Strength Material

Controlled low strength material (CLSM), or flowable fill, is a low strength concrete. Flowable fill is self-leveling, which allows it to fill voids (Puppala et al. 2009). CLSM backfill acts as a single unit when cured, which theoretically eliminates lateral wall stress (Snethen et al. 1997). Construction of a CLSM backfill on a highway embankment over the Salt Fork of the Arkansas River near Ponca City, Oklahoma was fast and simple. An advantage is that CLSM is resistant to erosion. There is concern that CLSM bonding to the abutment could increase the load on the foundation piles (Snethen et al. 1997, Amon et al. 1994). CLSM is useful in areas where compacting traditional fill is difficult because it is self consolidating (Lin et al. 2007).

2.7 IMPROVED CONSTRUCTION PRACTICES

2.7.1 Intelligent Compaction

The Federal Highway Administration defines intelligent compaction (IC) as using vibratory rollers equipped with a measurement and control system that can automatically control compaction parameters during compaction and a continuously recording documentation system (Rahman et al. 2007). IC rollers use either accelerometers or machine energy to calculate parameters related to modulus, stiffness, or bearing capacity. The information is then used to increase or decrease compactive effort (Camargo et al. 2006).

Rahman et al. (2007) studied the soil stiffness measured by IC rollers and correlated this to other stiffness measurement methods on two pavement reconstruction projects over sandy soils (SP or A-3). A conventional vibratory roller was used to compact a “proof” section. The IC roller was then passed over this and able to locate and record soft spots. It was observed that the IC roller stiffness is sensitive to field moisture content, and higher moisture contents will result in lower roller stiffness. Camargo et al. (2006) performed a case study of Minnesota Department of Transportation experience with intelligent compaction at three sites. Results from these sites were fairly uniform. A challenge currently present in IC methods is standardizing the data methods between companies and devices, and presenting the amount of data collected in a useful form.

Chapter 3 INVESTIGATION METHODS

3.1 INTRODUCTION

This chapter describes the processes and methods used in this investigation. The investigation included preliminary and in depth site visits, in situ testing, soil sampling, laboratory investigations, and a critical review of background information. Level I site visits were visual inspections of recommended bridges in order to gather information for the research as well as help determine bridges which merit further investigation. Level II investigations included subsurface soil sampling and in situ tests; the range of sampling and testing was determined individually for each site. Laboratory investigations sought to classify the soil and to describe soil behavior related to approach slab movement. Background data collected includes design and construction documents, as well as geologic, traffic, and other conditions that may be related to the embankment and approach behavior. The information gathered in the course of this investigation was stored in a database, which was used to statistically analyze trends relating bridge approach settlement to observed distresses. The details of the database are presented in the thesis by Osborne (2012).

3.2 SITE SELECTION

Site selection began with the OU investigation team working with ODOT. When the project was begun, ODOT representatives met with the principal investigators and a list of 12 sites was developed. The investigation team then sent surveys (see appendix) to resident engineers requesting information on bridges suffering from the bridge approach settlement issue in their residency.

As a result, 43 sites comprising 50 bridges were identified as potential investigation sites. These potential sites are tabulated in Appendix A. Once bridges were identified as potential sites, a Level I visit was made by the research team. These visits included a visual inspection of the approach and recording measurements of distress. Field visit observations were combined with information from plans, documents, and other sources to analyze trends related to bridge approach settlement and to select sites for Level II investigations. When selecting Level II sites for subsurface investigations, the goal of the team was to select sites with a variety of issues leading to approach settlement, geotechnical and structural designs, and approach conditions.

3.3 LEVEL I INVESTIGATIONS

Level I investigations involved visual inspection, manual measurements and creating a photographic record. Level I investigations were conducted at 23 different locations and involved 30 bridges. Table 3.1 provides a list of the sites visited. The visual inspection made note of the condition of the approach slab, abutment, embankment, wing walls, drainage and other peripheral structures for evidence of the causes of settlement and related issues. Where distresses were visible, the vertical and horizontal differences between the approach slab and the bridge deck and the approach slab and wing walls were measured with a ruler. In addition, where voids were visible beneath the roadway slab and other structures, such as slope walls, the depth of voids was measured in similar fashion using rulers and tape measures as needed. After each preliminary visit, the field team presented pictures and commentary to the entire research team at the weekly meetings. The team discussed each site regarding possible issues, general commentary, and whether a site was a potential candidate for additional subsurface investigations.

3.3.1 Measuring the Bump

This investigation used two methods to measure the “bump” at the end of the bridge. For measuring purposes, the bump was loosely defined as the vertical offset between the original and current position of the roadway surface. The first method was to use a ruler during the Level I investigations. There are two major concerns regarding this method. First, it did not account for maintenance performed which levels up the approach. Second, roads with heavy traffic were too dangerous for the investigation team to take any measurements. The second method is to account for maintenance. This is done by consulting maintenance records and by asphalt coring. Maintenance records do not always quantify how much leveling up was performed. Therefore, this measurement is inconsistently available. Bumps between 0” to 1” were classified as mild, 1”-2” as moderate, and greater than 2” as severe. Where measurements were incomplete, they are categorized either as “unclassified with maintenance” or “unclassified without maintenance.”

3.4 LEVEL II INVESTIGATIONS

Once a site was recommended for a Level II investigation, the bridge design sheets were consulted to determine the basic subsurface conditions. Based on the observed issues and subsurface conditions, subsurface investigations were specified for each embankment. The sites visited for Level II investigations are highlighted in Table 3.1. Level II investigations included drilling and sampling, laboratory testing and cone penetration soundings.

3.4.1 Drilling and Sampling

Drilling and sampling was conducted by the Oklahoma Department of Transportation using a truck mounted drilling rig and air rotary method. A representative from the OU research team was on site and assisted with sample logging. Split spoon and standard Shelby tube samples were taken, the latter primarily in cohesive layers. Drilling and sampling was conducted on and off the embankments. On the embankments boreholes were located near the abutment while off the embankment boreholes were located as close to the embankment as possible. Water content samples were collected in the field and immediately sealed in a container until weighing and oven drying could be accomplished in the laboratory. The standard Shelby tube method was used to obtain nominal 3-inch diameter thin-walled tube samples. Tubes were sealed at both ends and brought to the laboratory where they were generally extruded and processed for moisture room storage with one week.

3.4.2 Laboratory Testing

Laboratory testing was conducted in the OU soils laboratory to determine soil classification on selected samples while visual-manual classification was conducted on others. Classification testing generally encompassed Atterberg limit and grain-size distribution testing. In addition, oedometer tests were conducted on samples obtained from cohesive soils at four of the five test sites where cohesive soils were found. Oedometer testing was conducted on natural samples at the natural water content as well as on samples that were submerged (soaked). The idea was to look at the compressibility of the soil in an unsaturated state as well as volume change tendencies in the saturated state. Determining a range of compressibility parameters bracketing these states seemed a reasonable way to estimate settlements that may have already occurred in soil where the water content history is unknown. In addition, double oedometer tests were conducted on cohesive fill materials for sites where embankments reached 20 feet or more in height.

3.4.3 Cone Penetration Testing

Cone penetration soundings were conducted on and off the embankments and used to delineate stratigraphy at the test sites. The cone was equipped to measure tip resistance, sleeve friction, and pore water pressure (i.e. a piezocone). However, since the testing was conducted in predominantly unsaturated soils above the water table, the pore water pressure measurements were only used as an aid in determining the water table position where possible. Pushing the piezocone through a significant zone of unsaturated soil can introduce air into the sensing element, especially in strongly dilative soils, which affects its response time and reliability. In addition, pore pressure readings in unsaturated soils are not subject to interpretation by available methods.

3.5 COLLECTION OF BACKGROUND DOCUMENTATION

3.5.1 Bridge Design, Construction and Maintenance Records

Once potential sites were selected, requests were sent to engineers from the ODOT Planning and Research Division for the bridge drawings and maintenance records by providing the bridge location and National Bridge Inventory (NBI) number. Documents requested include bridge sheets containing general plan and elevation, foundation reports, structural design and construction, material and maintenance records.

3.5.2 ODOT Geographical Resource Internet Portal Lite

The ODOT Geographical Resource Internet Portal Lite (GRIP Lite) service was used to match the bridge sites and NBI numbers provided by ODOT engineers with the correct geographical location. The GRIP Lite system also provided information for each bridge including the 2008 average daily traffic (ADT), construction date, as well as basic dimensions, geometry and structural details.

3.6 DATABASE DEVELOPMENT AND ANALYSIS

The first step in relating approach settlement problems to design and/or construction factors was identifying specific problems at each bridge observed during Level I investigations. Then, likely mechanisms of the cause were investigated in the design documents and other background data, depending on the nature of the problem. Relationships were investigated by means of inductive reasoning where general lessons were learned by investigating specific problems. When bridges had similar issues, the background data was compared to investigate the possibility of trends, and strengthen the evidence for relationships between approach design and the settlement problems observed. This was done by querying the database. Database development is detailed in Osborne (2012).

Table 3.1 List of Sites Subjected to Level I and Level II Investigation

Item	Site/Intersection	Over	County	ODOT Division
1	SH-33 over Fitzgerald Creek	Water	Logan	4
2	Shields Blvd. over I-35	Road	Cleveland	3
3	US-62 over Robinson Creek	Water	Lincoln	3
4	SH-3W over Big Creek	Water	Pontotoc	3
5	SH-48A over Blue River	Water	Johnston	3
6	SH-59B over Coon Creek	Water	Pottawatomie	3
7	SH-11 over I-35	Road	Kay	4
8	US-177 over Salt Fork River	Water	Noble	4
9	I-44 over Medicine Bluff Creek	Water	Comanche	7
10	SH-7 over Beaver Creek	Water	Comanche	7
11	SH-6 North of Retrop	Water	Washita	5
12	SH-6 over Sadler Creek	Water	Beckham	5
13	SH-6 over West Elk Creek	Water	Beckham	5
14	SH-1 over BNSF Railroad	Road	Pontotoc	3
15	SH-1 over SH-99	Road	Pontotoc	3
16	SH-59A over Big Creek	Water	Pontotoc	3
17	19th Street over I-35	Road	Cleveland	3
18	I-35 over Main	Road	Cleveland	3
19	Tecumseh Road over I-35	Road	Cleveland	3
20	SH-152 over Lake Creek	Water	Caddo	7
21	SH-152 over Lake Creek Overflow	Water	Caddo	7
22	SH-152 over Willow Creek	Water	Caddo	7
23	SH-9 over Running Creek	Water	Caddo	7

Note: Shaded rows designate sites subjected to Level II investigation

Chapter 4 RESULTS OF LEVEL I INVESTIGATION

4.1 SUMMARY OF OBSERVATIONS

The inspection team conducted Level I investigations on 30 bridges at 22 locations listed in Table 3.1. These observations were analyzed to investigate trends between bridge design and construction and bridge approach settlement issues. Erosion and drainage failure were common issues observed during Level I investigations. Fifteen bridges were identified as representative examples of the erosion and drainage concerns observed. Comparing the observed conditions to drainage design at selected sites revealed possible solutions for improving drainage design.

In addition to Level I investigations, the scope of this research project included choosing specific bridges based on a number of criteria to perform subsurface investigations. Five bridges were identified and selected for subsurface investigations. Level II investigations consisted of subsurface investigations and were conducted in order to better determine the cause of the noted problems, namely embankment settlement, erosion and foundation soil issues.

4.2 EROSION PROBLEMS

Erosion and drainage issues were a common finding throughout the state during Level I investigations. Substantial voids were observed beneath the approach slabs, abutments and slope walls. Voids under the approach slab undermine the support and can result in slab settlement and structural damage. The most common erosion paths observed during this investigation were under the abutment, laterally from directly beneath the approach slab, and through cracks and joints in the abutment. Many sites use rip-rap or concrete slope walls obscuring visual observation of the abutment; in some cases, definitive comments about erosion cannot be made. The common flow paths are illustrated in Figure 4.1 using a typical abutment design.

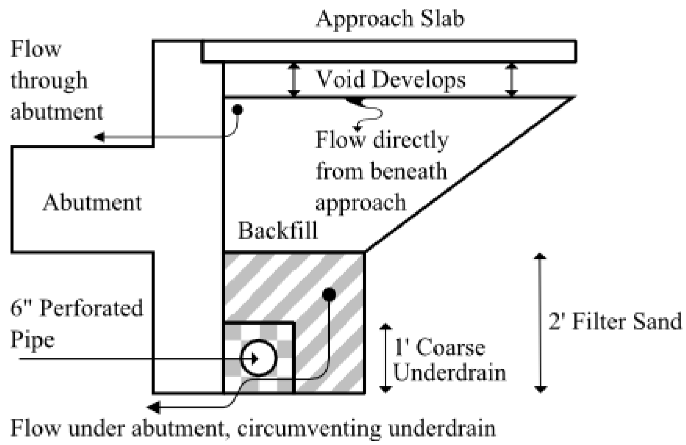

Figure 4.1 Diagram of Most Significant Erosion Paths

Fifteen bridges with erosion problems are presented as examples in this chapter; some bridges listed here coincide with bridges selected for subsurface investigations while other bridges do not. A summary of the example bridges with erosion and drainage issues is presented in Table 4.1. This table briefly describes the condition of the surface drainage and drainage outlet, whether there is any flow through cracks in the abutment, the severity of the bump, the presence of voids and observed erosion paths.

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Table 4.1 Summary of Erosion and Drainage Problems by Bridge

Bridge Location	Drainage Condition	Issues
S.H. 6 over Sadler Creek	***Surface**:* Joints between wing-wall and backfill and between pavement slabs are open. **Outlet**: Open ***Abutment**:* Traces of soil stains on abutment	***Bump***: Mild ***Voids***: Severe; 1' under the approach slab, piles exposed under the abutment ***Erosion Paths***: Side of approach
S.H. 6 over West Elk Creek	***Surface***: Joints between wing-wall and backfill and between pavement slabs are open. ***Outlet***: Not found ***Abutment***: Not observed	***Bump***: Severe, up to 3" ***Voids***: 0.5-4" under the slab, up 4" under abutment ***Erosion Paths***: Obscured by rip-rap
U.S. 177 over Salt Fork, A	***Surface***: Pavement joints open, surface cracked, separation between approach and wing wall. ***Outlet***: Open, dry ***Abutment***: Soil flow stains on abutment	***Bump***: 2" plus asphalt overlays ***Voids***: 6" under abutment, piles are exposed ***Erosion Paths***: Under abutment
U.S. 177 over Salt Fork, B	***Surface***: Joint between approach and bridge open and filled with debris, cracked surface, separation between wing-wall and approach ***Outlet***: Not found ***Abutment***: Stains visible	***Bump***: 2" ***Voids***: 1'9" under abutment ***Erosion Paths***: Under abutment
S.H. 152 over Lake Creek Overflow	***Surface***: Cracks in approach surface ***Outlet***: No underdrain ***Abutment***: Some red staining on abutment wall.	***Bump***: New asphalt ***Voids***: Large voids under the abutment. ***Erosion Paths***: Under abutment
S.H. 152 over Lake Creek	***Surface***: Approach surface cracked, separation between wing-wall and approach slab. ***Outlet***: No underdrain ***Abutment***: None observed	***Bump***: 2.5" bump, new asphalt ***Voids***: Large voids under abutment ***Erosion Paths***: Under abutment
S.H. 152 over Willow Creek	***Surface***: Approach surface cracked, separation between wing-wall and approach slab. ***Outlet***: No underdrain ***Abutment***: Red staining near outside	***Bump***: 2.5" bump, new asphalt ***Voids***: Large voids under abutment ***Erosion Paths***: Under abutment

Bridge Location	Drainage Condition	Issues
S.H. 9 over Running Creek	***Surface***: Approach surface severely cracked, ***Outlet***: No underdrain ***Abutment***: No evidence observed	***Bump***: Moderate 1.5" Bump ***Voids***: Large voids under the slab and abutment ***Erosion Paths***: Under abutment
S.H. 59 A over Big Creek	***Surface***: Cracked surface, separation between wing-wall and approach ***Outlet***: Not found ***Abutment***: No stains observed	***Bump***: 1.5"; Asphalt overlay ***Voids***: 7" under slab ***Erosion Paths***: Side of abutment
Tecumseh Road over I-35	***Surface***: Large separation between approach slab and wing-wall. Pavement joints no longer sealed. Cracked approach slab. ***Outlet***: Apron outlet flows right into unsealed joint and erodes soil. ***Abutment***: Water only across front, no evidence of soil	***Bump***: Mild ***Voids***: Up to 17" under slab. ***Erosion Paths***: Under slope wall
S.H. 11 over I-35	***Surface***: Cracks in the approach, embankment drain caved in. ***Outlet***: Not found ***Abutment***: Inconclusive stains	***Bump***: Mild ***Voids***: None observed ***Erosion Paths***: None observed
Shields Boulevard over I-35	***Surface***: Cracked surface, open pavement joints, separation between approach and wing-wall ***Outlet***: Some functioning, some buried, some have soil flow exiting ***Abutment***: No stains	***Bump***: Moderate ***Voids***: Under apron and abutment ***Erosion Paths***: Through underdrain pipe
19th St. over I-35	***Surface***: Sealed joints unsealed, cracked approach slab, ***Outlet***: Buried ***Abutment***: Water flow, potentially soil flow	***Bump***: Mild ***Voids***: None-observed ***Erosion Paths***: None-observed
I-35 over Main St. (Moore, OK)	***Surface***: Cracked approach ***Outlet***: Buried ***Abutment***: None observed	***Bump***: No measurement ***Voids***: None observed Erosion Paths:

Bridge Location	Drainage Condition	Issues
S.H. 3W over Big Creek	***Surface***: Good condition ***Outlet***: Not found ***Abutment***: No soil flow observed	***Bump***: None ***Voids***: Beneath abutment ***Erosion Paths***: Beneath abutment

Erosion and settlement issues often were observed at sites with failed drainage systems. Surface drainage was universally poor. In some cases, joints between slabs or between the approach slabs and wing-walls had separated and were no longer sealed, approach slabs also cracked and allowed water into the backfill, and concrete channels designed to transport water away from embankments had collapsed into voids. Figure 4.2 shows the percentage of all investigated embankments with each drainage issue observed. It should be noted that in many cases, the outlet could not be located; therefore, the "outlet blocked" statistic may be underreported.

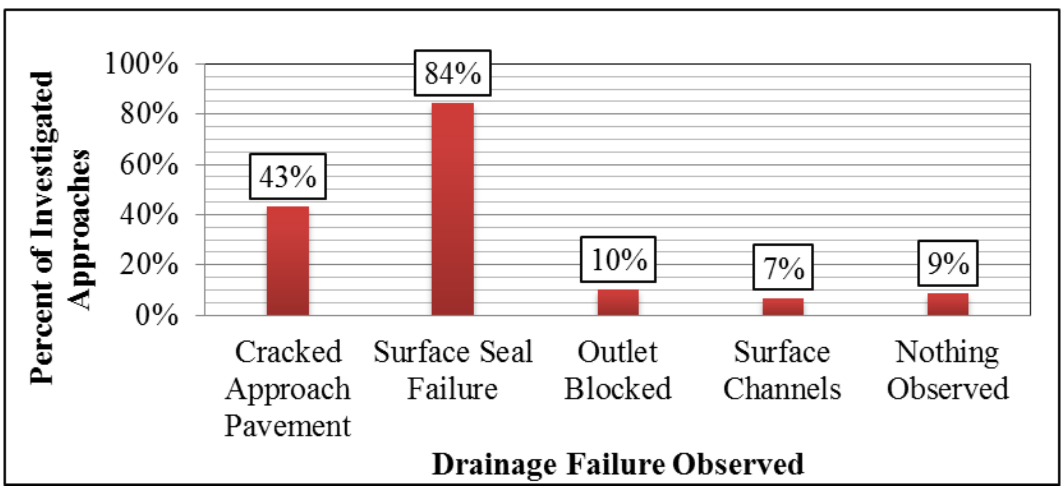

Figure 4.2 Percentage of investigated approaches with drainage issues (Total approaches: 58)

Comparisons were made between drainage conditions and bump distress. Mild bumps are those measuring less than one inch, moderate bumps are between one and two inches and severe bumps are those measured at greater than two inches. Where difficulty was encountered measuring the bump, the approaches are "unclassified." A distinction is made regarding whether any maintenance was observed during this research or recorded in ODOT bridge inspection reports. This was done in order to have at least some indication of the extent of the problem at the unclassified approaches. Figure 4.3 illustrates the distribution of bump severity at approaches identified as having failed surface pavement joint seals.

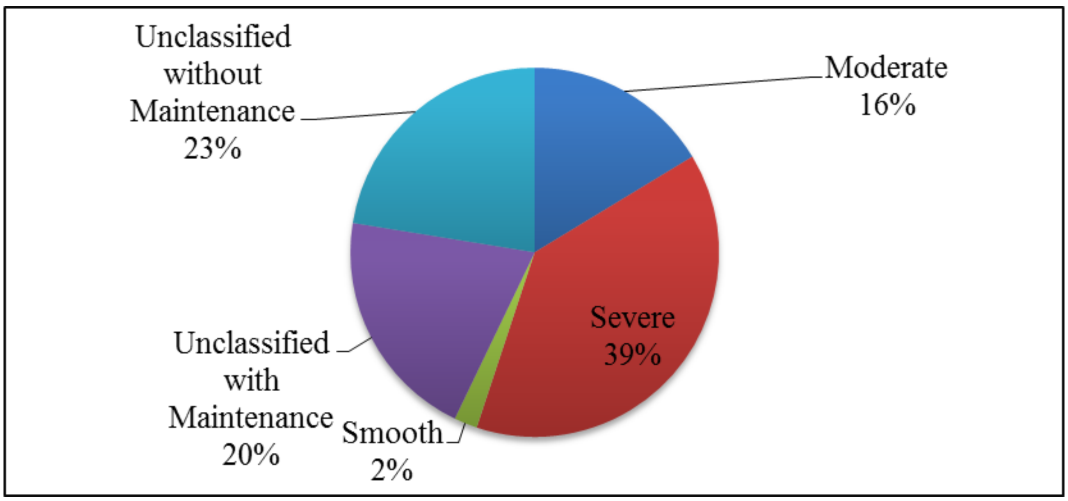

Figure 4.3 Bump severity at approaches with failed pavement joint seals (49 approaches)

Examples of common problems are seen in Figure 4.4. A large gap can be seen between the wing wall, moving outwards, and the approach slab on Tecumseh Road over I-35. The developing bump on S.H. 6 over West Elk Creek has pulled the seal between pavement slabs apart. A concrete surface channel designed to carry water away from the embankment on S.H. 11 over I-35 failed and collapsed into a 3' deep void beneath the channel.

(a) Horizontal separation between wing wall and approach slab

(b) Vertical separation destroys seal

(c) Surface drain channel caving into 3' deep void

Figure 4.4 Failure of surface drainage

Some examples of erosion taking place through the abutment are shown in Figure 4.5. Soil stains could be observed on the abutment wall originating in these cracks suggesting that some soil loss is taking place through these cracks.

(a) Water and soil flowing through a cracked abutment on S.H. 59 B over Coon Creek, photographed May 21st, 2011

(b) Red stains on the abutment indicate the possibility of soil loss through U.S. 177 over the Salt Fork, photographed June 2nd, 2011

(c) Red staining on abutment at S.H. 152 over Lake Creek Overflow, photographed November 11th, 2011

(d) Red stains originating from crack at S.H. 1 over BNSF, photographed August 26th, 2011

Figure 4.5 Water and soil flowing through cracked abutments

Voids between the approach slab and the earth were common, but the mechanisms creating this void varied. A void underneath the approach slab creates a loss of support that can directly lead to bridge approach settlement and a bump. Voids were also common under the abutment. Besides exposing the piles, voids underneath the abutment increase erosion potential of the backfill and can lead to voids developing underneath the approach slab. The percentage of approaches with voids present is presented in Figure 4.6.

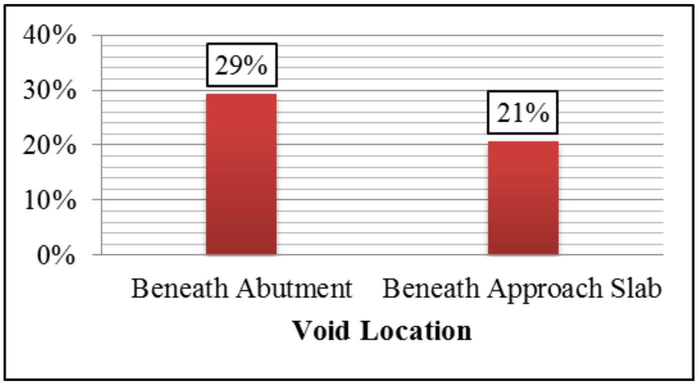

Figure 4.6 Percentage of approaches with voids (Total approaches: 58)

Once voids were present under the approach slab, soil was able to flow laterally directly from beneath the approach slab, as seen at S.H. 6 over Sadler Creek in Figure 4.7. An example of a void under the approach slab is shown at S.H. 59 A over Big Creek in Figure 4.8. An example of an under-abutment void is shown at U.S. 177 over Salt Fork, Bridge B in Figure 4.9.

Figure 4.7 Soil flowing laterally from beneath approach slab

Figure 4.8 A 7" void under approach slab

Figure 4.9 A 1’ 9” void under abutment

4.2.1 Underdrain Design and Site Behavior

A variety of underdrain designs were observed for each of the 30 bridges investigated during the Level I site visits, with varying results. The standard ODOT underdrain design is specified for integral abutment bridges in sheets B40-I-ABUT-MISC 01E (2009) and 00E (1999), *Substructure Excavation and Pipe Underdrain Assembly Details Integral* and for conventional abutment bridges in sheets B40-C-ABUT MISC 01E (2009) and 00E (1999), *Substructure Excavation and Pipe Underdrain Details Conventional*. Of the bridges identified in this section, none uses the standard exactly; instead, many are similar but modified while others vary significantly from the standard. The major change between 1999 and 2009 is the addition of “filter fabric” being wrapped around the 1’ coarse cover material. A concern with the standard design is that there is no mechanism to prevent water from flowing under the abutment and transporting soil material. The standard underdrain design is illustrated in Figure 4.10.

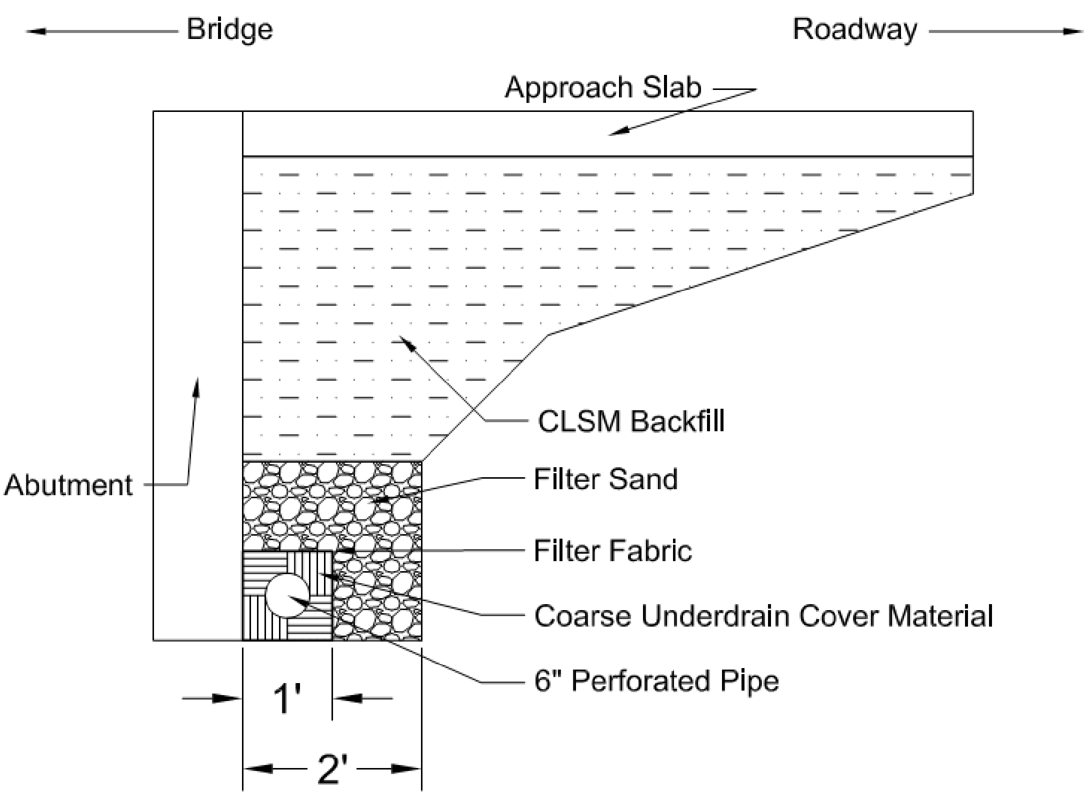

Figure 4.10 Standard ODOT underdrain design, after ODOT 2009 Integral Bridge Design Standards & Specifications

4.2.2 Examples of Erosion Beneath the Abutment

Erosion under the abutment was a common observation during Level I investigations. Four bridges in Caddo County, S.H. 152 over Lake Creek, S.H. 152 over Lake Creek Overflow, S.H. 152 over Willow Creek and S.H. 9 over Running Creek experienced this problem. No underdrain was specified at these locations. An erosion channel at S.H. 152 extending from beneath the abutment to Lake Creek is shown in Figure 4.11 and Figure 4.12.

Figure 4.11 Erosion channel under abutment at S.H. 152 over Lake Creek

Figure 4.12 Erosion channel at S.H. 152 over Lake Creek continues

A similar channel was observed at S.H. 152 over Willow Creek and is shown in Figure 4.13. Expanding foam maintenance was used beneath S.H. 9 over Running Creek, but the erosion continued and a void is present even below the foam as seen in Figure 4.14. An example of erosion beneath the abutment is presented in Figure 4.15.

Figure 4.13 Erosion channel from beneath abutment at S.H. 152 over Willow Creek

Figure 4.14 Void and maintenance under abutment at S.H. 9 over Running Creek

Figure 4.15 Erosion flow beneath abutment at S.H. 152 over Lake Creek Overflow

State Highway 3W over Big Creek is a conventional abutment bridge with an underdrain design fairly similar to the 1999 standard. The major difference is that, instead of having a filter sand layer and a coarse pipe underdrain material layer, there is only 2’ of filter sand surrounding the pipe. Also, the top of the filter material is level with the bottom of the abutment. The approach performance at this site is acceptable. No bump was observed, and the surface structure was in good shape when the team visited on May 21st, 2011, as seen in Figure 4.16. There was however, a void developing under the abutment, as seen in Figure 4.17. The underdrain design for this site is shown in Figure 4.18.

Figure 4.16 The driving surface is smooth on S.H. 3W over Big Creek

Figure 4.17 Void forming beneath abutment at S.H. 3W over Big Creek

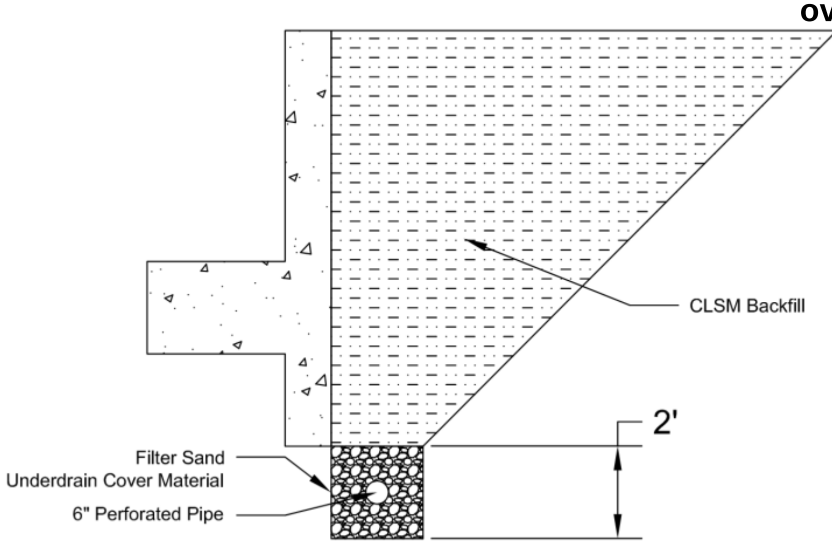

Figure 4.18 Underdrain design, after ODOT Proposed Plan for S.H. 3W over Big Creek

S.H. 6 over West Elk Creek features a design somewhat similar to the 1999 standard for integral bridges. The most significant difference is that granular backfill, as opposed to CLSM, is specified. The driving surface at this site is rough. A 3” bump was measured between the bridge and the north approach. Voids up to 4” were measured both under the approach slab and beneath the abutment. These findings are presented in Figure 4.19 through Figure 4.22. The underdrain design is shown in Figure 4.23.

Figure 4.19 A 3" bump

Figure 4.21 4" void beneath the approach slab

Figure 4.20 4" void beneath the abutment

4.22 Void beneath north abutment

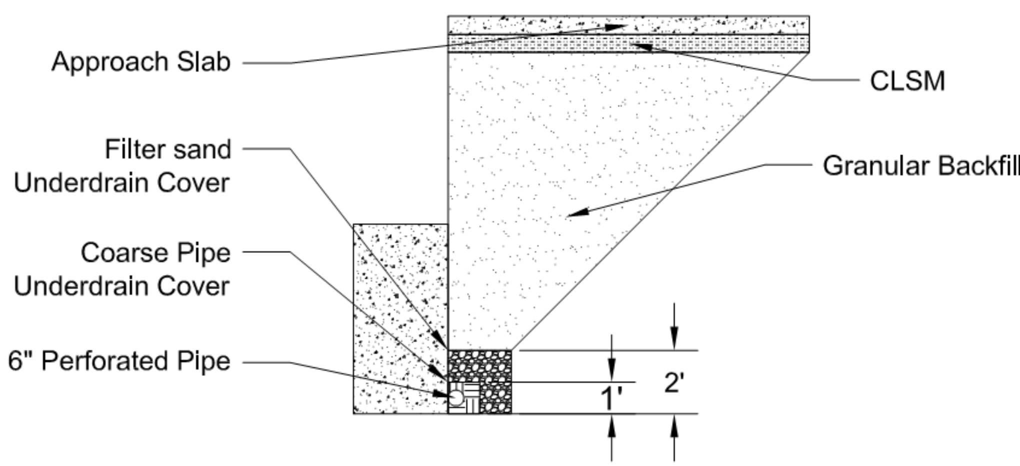

Figure 4.23 Underdrain Design, after ODOT Proposed Plan for S.H. 6 over West Elk Creek

U.S. 177 over the Salt Fork consists of three bridges, each with different embankment construction techniques. Embankment A-2 was constructed using typical construction practices. A bump developed at this approach and the asphalt is cracked. Erosion was observed beneath the abutment. Figure 4.24 shows the approach A-2 and Figure 4.25 shows the observed erosion. The underdrain design is shown in Figure 4.26.

Figure 4.24 US 177 Approach A-2

Figure 4.25 Erosion under the abutment on US 177 Embankment A-2

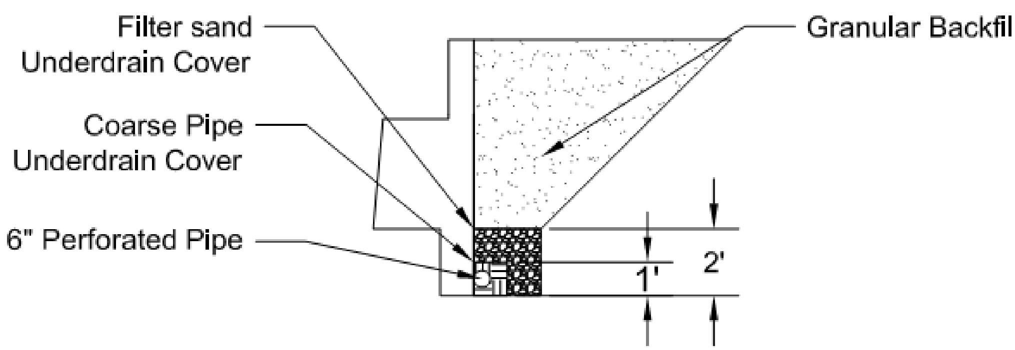

Figure 4.26 Underdrain Design, after ODOT Proposed Plan US 177 over Salt Fork, Bridge A

Embankment B-1 utilized geotextile reinforcement. A 2" bump was measured at this location, as seen in Figure 4.27. Substantial erosion under the abutment was observed and is shown in Figure 4.28. The underdrain design is shown in Figure 4.29.

Figure 4.27 Bump at U.S. 177 over Salt Fork, Approach B-2

Figure 4.28 Erosion beneath abutment at U.S. 177 over Salt Fork, Embankment B-2

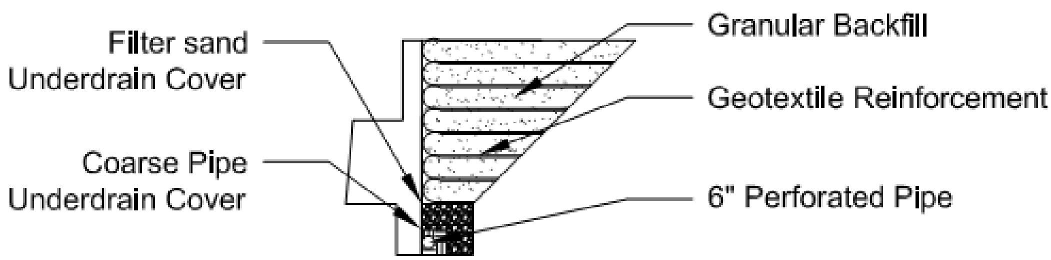

Figure 4.29 Underdrain Design, After ODOT Proposed Plan U.S. 177 over Salt Fork

4.2.3 Examples Without Erosion Beneath the Abutment

Other underdrain designs appeared to prevent erosion from occurring under the abutment. Two notable examples of designs which prevented under-abutment flow are Shields Boulevard over I-35 and Tecumseh Road over I-35.

Shields Boulevard over I-35 utilizes an underdrain design that appears to prevent erosion beneath the abutment. A neoprene sheet is specified that prevents flow between the abutment and the abutment footing. The 6" perforated pipe is surrounded by coarse pipe underdrain material which is in turn surrounded by filter sand; no filter fabric is specified. Granular backfill is specified. The underdrain design is shown in Figure 4.30.

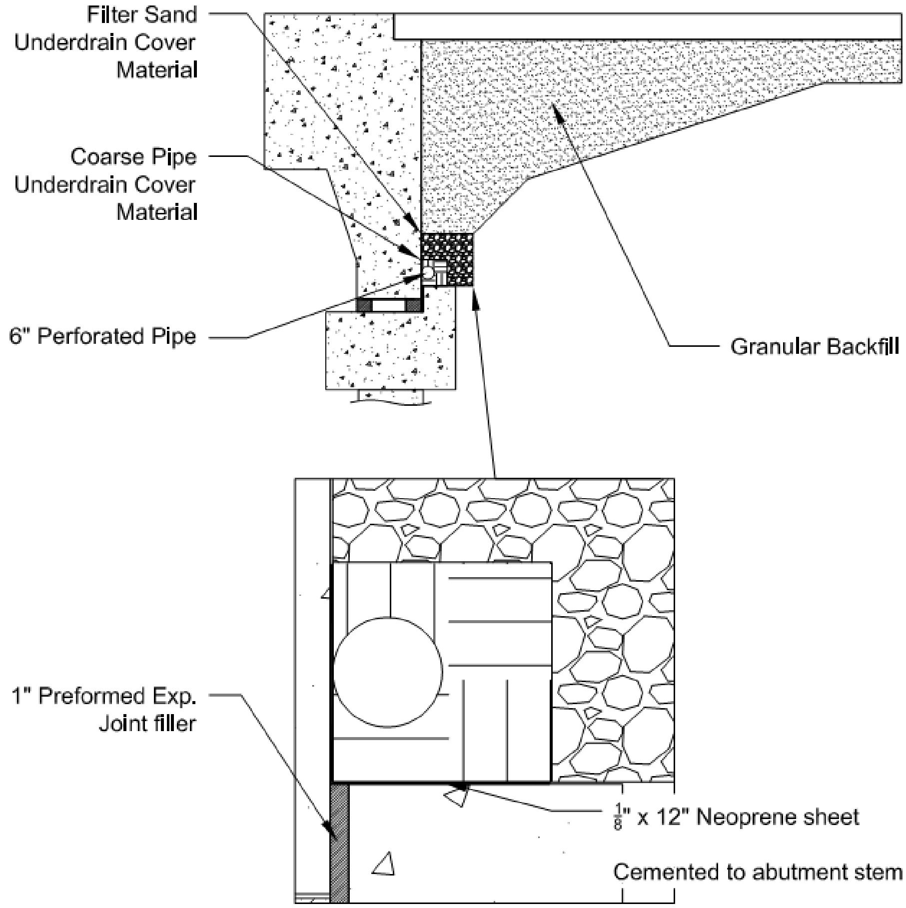

Figure 4.30 Underdrain Design, After ODOT Proposed Plan for Shields Boulevard over I-35

A concrete slab extends beneath the abutment and underdrain at Tecumseh Road over I-35. No erosion beneath the abutment has been observed at this site; however, asphalt had been poured into voids near the side of the abutment. There is possibility that water is flowing out of the side of the abutments. The general underdrain section is shown in Figure 4.31.

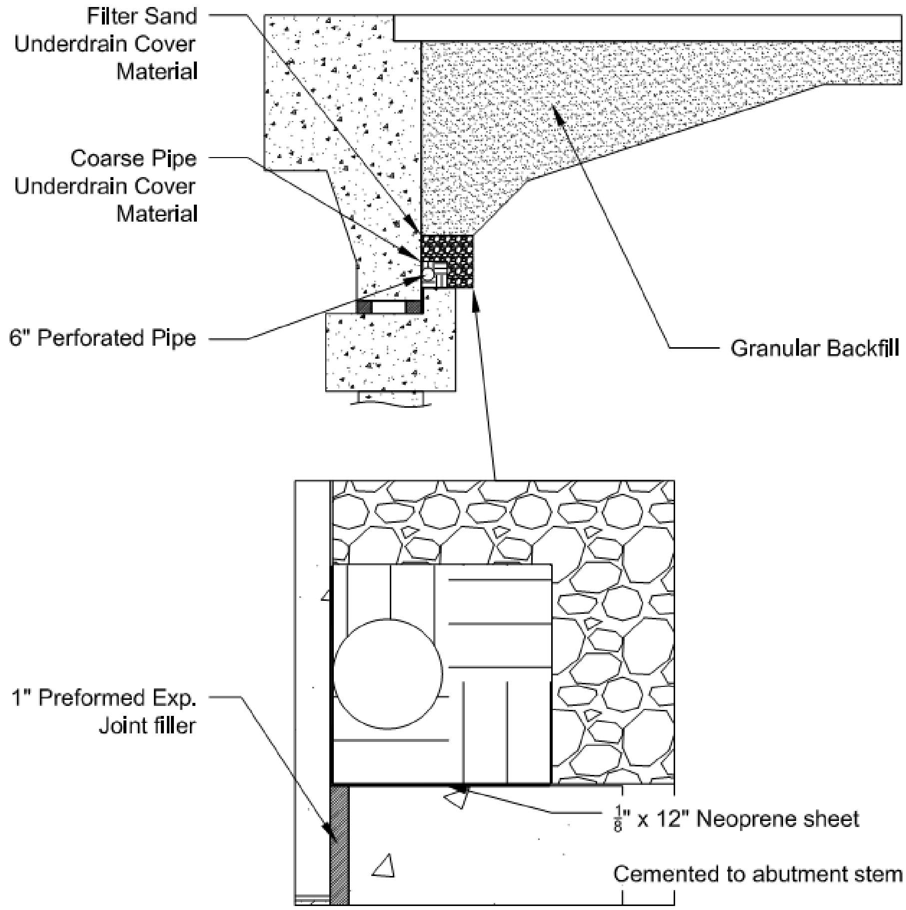

Figure 4.30 Underdrain Design, After ODOT Proposed Plan for Shields Boulevard over I-35

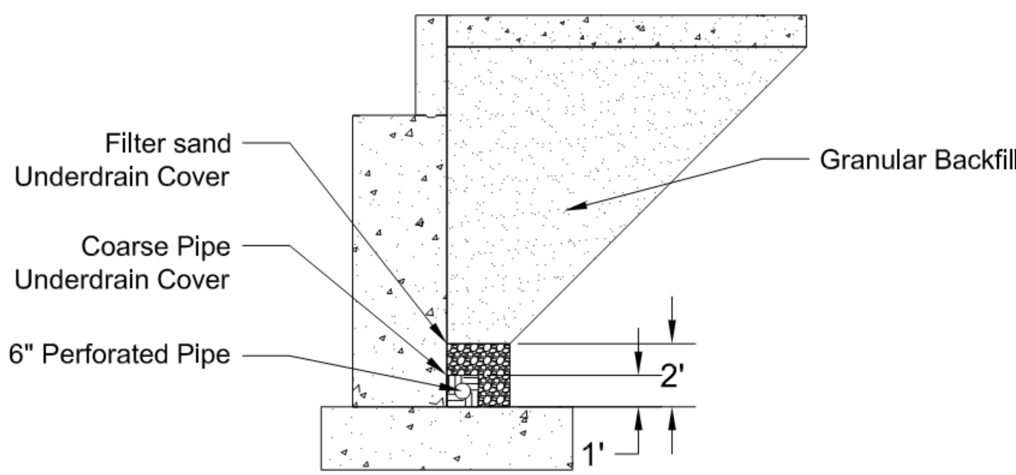

Figure 4.31 Underdrain Design, After ODOT Proposed Plan for Tecumseh Road over I-35

4.2.4 Examples of Poor Drainage Filtering

Another observed issue was soil being transported through drainage pipes. This indicates that improper filtering is allowing soil into the perforated pipes which are present in backfills and in slope walls. Shields Boulevard over I-35 provides an example of this. The pipes at this site are not only carrying water, but soil. Soil was observed to have flowed out of the outlets at this location, and voids were present at the abutment. This location has 8”-12” of settlement at the wing-walls, distress of the structure and approach slab, and a significant bump. The drainage outlet at the bottom of the slope wall is shown in Figure 4.32 and a developing void beneath the slope wall at the foot of the abutment is shown in Figure 4.33.

Figure 4.32 Soil flowing from underdrain outlet

Figure 4.33 Void forming beneath slope wall and abutment

4.2.5 Examples of Poor Drainage Outlet Placement

In some cases, the underdrain outlets were expelling water. However, the water was allowed to flow back into the embankment near the abutment. Two outlets at Tecumseh Road over I-35 were placed near the top of a slope wall, approximately 11’ in front of the abutment. The outlets were just above a joint in the slope wall. It is unclear which outlet serves the backfill and which serves the slope wall. Outlets are only on this side. The seal between the slabs had failed and therefore the water was poured directly back onto the embankment leading to erosion. The outlet and resulting 16” void is shown in Figure 4.34.

Figure 4.34 Water flowing from outlets into embankment, Tecumseh Rd. over I-35. (note large void under slope wall)

A similar situation was observed at Shields Boulevard over I-35. Though there were no cracks in the slope wall, the water flowed off the side of the slope wall. This led to water reentering the embankment and erosion. Figure 4.35 shows water flowing back into the embankment.

Figure 4.35 Outlet drain at Shields Boulevard over I-35, Embankment N-2

4.2.6 Examples of Non-Functioning Drainage Outlets

In many locations, the outlet drain is buried or crushed. The underdrain is useless if water cannot flow out of the outlet. Water becoming trapped in the embankment could have detrimental effects on the embankment performance. In Moore, bridges carrying 19th St. over I-35 and I-35 over Main Street have drains which are buried and blocked. No significant erosion was observed at either bridge, but the embankments were observed to have settled, as shown in Figure 4.36.

(a) Grass and soil blocks drainage outlets at 19th St. over I-35

(c) The outlet at I-35 over Main St. is almost completely buried

(b) Blue lines on the wing-wall indicate the original soil level and the settlement of the embankment relative to the structure at 19th St. over I-35

(d) 2” of embankment settlement is measured between the embankment and wing-wall at I-35 over Main St.

(e) 2” vertical separation is measured between the pavement and bridge interface at I-35 over Main St.

Figure 4.36 Photographic examples of buried drainage and embankment settlement

4.3 A POSSIBLE DESIGN STRATEGY TO MITIGATE EROSION UNDER ABUTMENTS

One of the most significant observations of this study is the amount of erosion occurring under abutments. This likely is contributing to significant loss of backfill and approach slab settlement. These observations led to a careful scrutiny of the backfill drainage design details for abutments, which resulted in an important finding. That is, basically some of the design details, such as those similar to the standard detail for integral abutment bridges in Figure 4.10, allow for a hydraulic short circuit to develop beneath the abutment. Thus, water in the backfill can largely bypass the drainage pipe and exit beneath the abutment causing large amounts of erosion over time. Figure 4.37 provides a schematic of the problem. The problem is that the proper function of the drainage system in Figure 4.10 requires that the contact between the bottom of the abutment and the underlying soil remains water tight so that water will be contained in the drain and exit via the pipe. Since the abutment is supported on piles it will generally not settle. However, any small amount of settlement of the soil beneath the abutment, which is highly likely in most cases, will cause a gap to open up and allow water to bypass the drain. Even if the embankment soil does not settle, it is unlikely that this contact is water tight and eventually water will begin to flow through this interface and cause erosion. Over time, extensive erosion can occur as observed at many sites.

A modification to this design is needed to introduce a water stop at the interface of the abutment and underlying soil. Figure 4.30 shows an example of a conventional abutment design where a water stop appears to have worked properly. Four alternatives to the design shown in Figure 4.10 are presented in Figures 4.38 to Figure 4.41. The essential element in these figures is the water stop, in the form of a geomembrane, that prevents water from escaping under the abutment. Figures 4.40 and 4.41 place the drain below the bottom of the abutment for added insurance that water will preferentially flow into the drain. Other differences noted in the figures are the use of a granular filter material (Figures 4.38 and 4.40) versus a geotextile filter (Figures 4.39 and 4.41) to protect the coarse cover material surrounding the drain pipe. The geomembrane ideally should exhibit high elongation under load, such as found with high density polyethylene or PVC to accommodate stretching forces that may occur as the drain settles relative to the abutment. That is, friction between the geomembrane and concrete may resist the downward movement of the geomembrane if the soil underlying the drain settles. A geomembrane with high elongation potential should be able to accommodate this action. Furthermore, it would be desirable to minimize the friction between the geomembrane and concrete abutment to allow the geomembrane to simply slip downward if the underlying soil settles. Of course, the height of the geomembrane on the abutment side (i.e. the vertical overlap against the abutment) should be large enough to allow some downward movement of the drain relative to the abutment without compromising the water stop.

The proposed drainage details presented in Figures 4.38 to 4.41 were developed with input from the ODOT Materials and Bridge Divisions. They can be modified to accommodate other backfill materials such as Controlled Low

Strength Material (CLSM) and other abutment configurations provided the guiding principles discussed above are followed. The CLSM is seeing more use as backfill behind abutments and may be preferable to granular fill in terms of quality control concerns related to compaction of granular backfill. The guiding principles outlined above should be followed in addition to the standard principles of proper drain function with regard to filtration and flow rate. In addition, added details will be needed to deal with the end regions of the drainage system near the wing walls and pipe drain outlets.

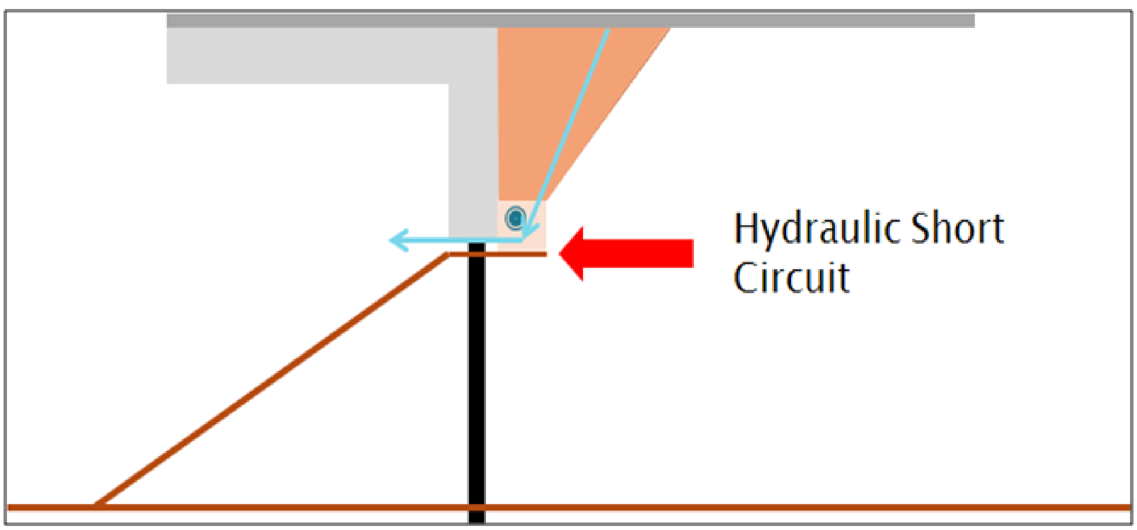

Figure 4.37 Schematic Cross-Section View of Hydraulic Short Circuit that Can Develop in the Abutment Drainage System

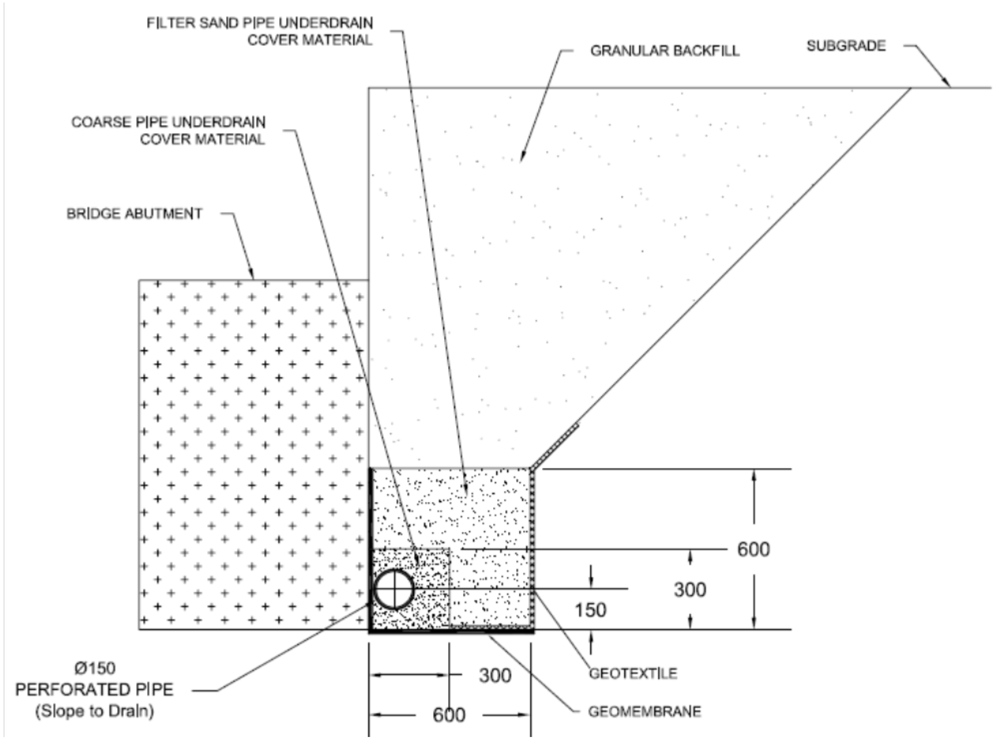

Figure 4.38 Proposed Abutment Backfill Drainage System with Geomembrane and Graded Granular Filter (units in mm: 300 mm≈12 in.)

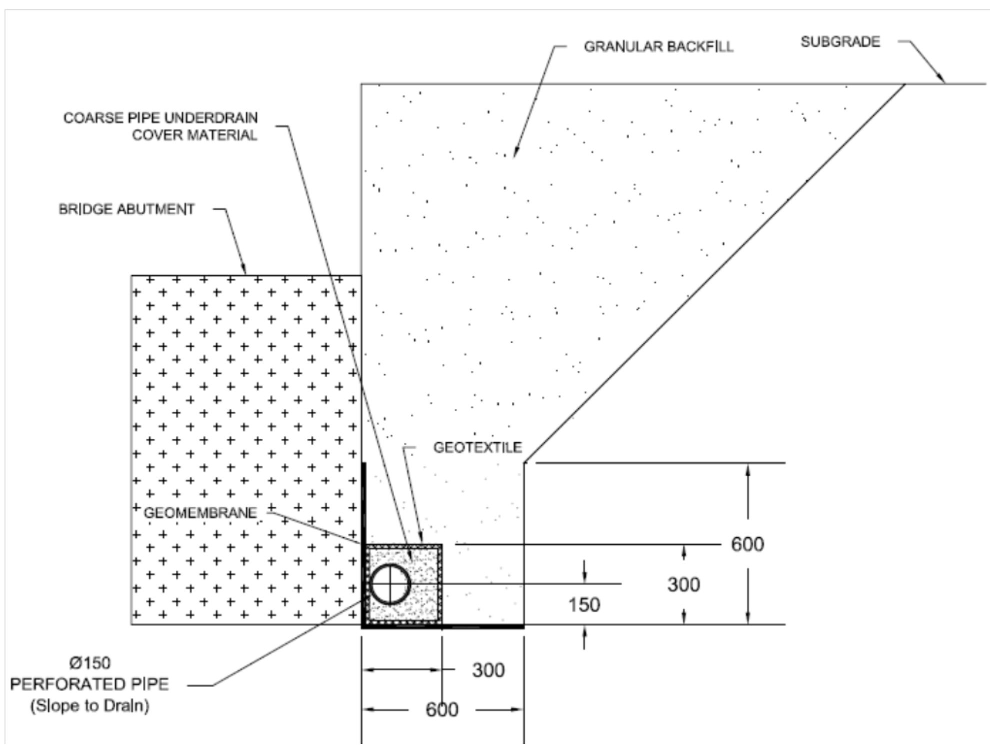

Figure 4.39 Proposed Abutment Backfill Drainage System with Geomembrane and Geotextile Filter (units in mm: 300 mm≈12 in.)

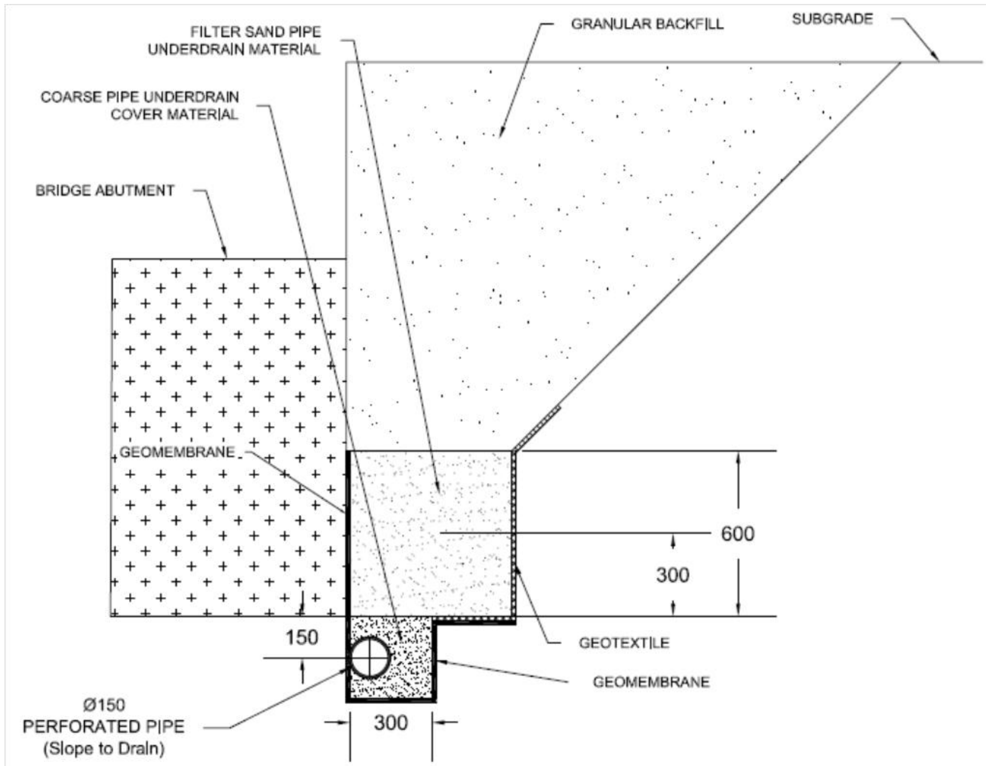

Figure 4.40 Proposed Abutment Backfill Drainage System, Depressed with Geomembrane and Graded Granular Filter (units in mm: 300 mm≈12 in.)

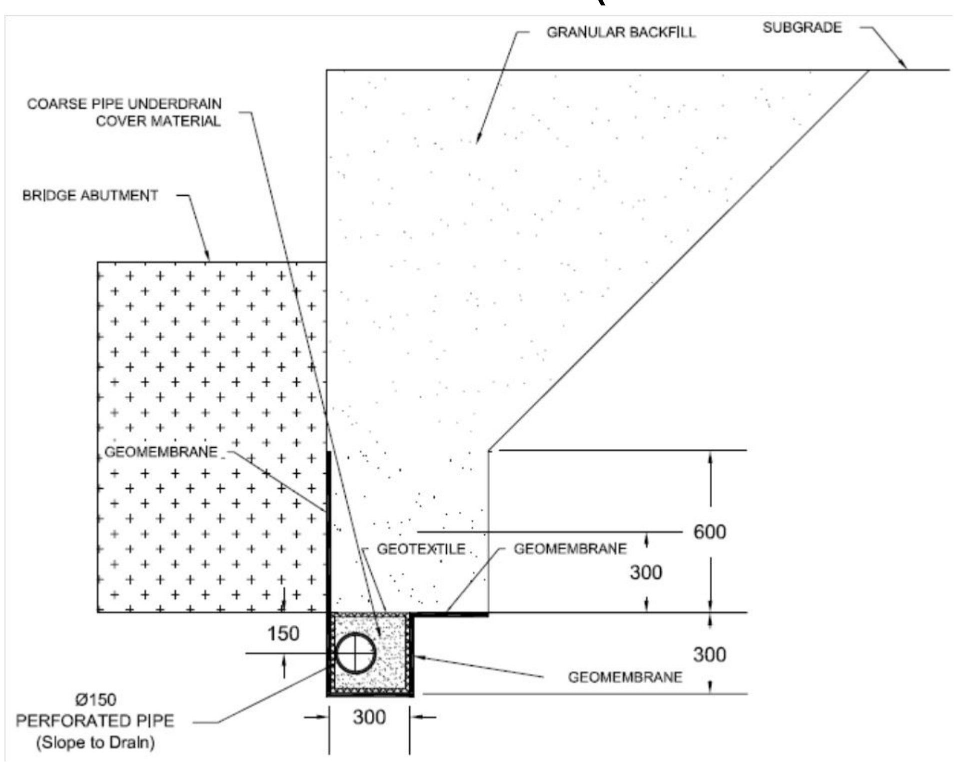

Figure 4.41 Proposed Abutment Backfill Drainage System, Depressed with Geomembrane and Geotextile Filter (units in mm: 300 mm≈12 in.)

Chapter 5 RESULTS OF LEVEL II INVESTIGATIONS

5.1 INTRODUCTION

Five sites were subjected to a detailed subsurface investigation to further reveal the underlying causes of approach slab settlement. The sites represent integral and non-integral conventional abutments, various embankment heights and a variety of subsurface soil conditions. At each site, additional subsurface work included drilling and sampling, cone penetration testing, and laboratory testing. Additional analysis for some sites included evaluation of settlement. The five sites selected for in-depth investigation are identified in Table 3.1.

5.2 SH-59A OVER BIG CREEK

This integral abutment bridge was constructed in 1996. Maintenance reports indicated that 3-4” of approach settlement had occurred by 1998, and the issue continued to worsen in subsequent reports. Asphalt overlay maintenance had been performed at some point, but generally the approach condition is poor. In addition, during the initial site visit by the research team, voids were observed beneath the approach slab and there was evidence of water seeping through cracks in the abutment. Table 5.1 presents a summary of information on this bridge.

5.2.1 Results of Subsurface Investigation

Three test borings and three cone soundings were conducted, both on and off the west embankment as shown in Figure 5.1. Test boring logs and tabulated soil properties are contained in Appendices B and C, respectively. Soil properties and interpreted soil profiles are shown in Figure 5.2 and results of cone soundings are shown in Figure 5.3. Below about 7 feet of sand fill, borings and cone soundings encountered predominantly fine-grained clayey soils with moderate to high plasticity extending 60 feet below the sand fill, and above shale. Normalized undrained shear strengths (c_u/σ'_{vo}) interpreted from CPT results, shown in Figure 5.2, generally indicate the clayey soils are lightly to moderately overconsolidated, with occasional highly overconsolidated stiffer layers. For reference, normally consolidated to lightly overconsolidated, non-structured, sedimentary clays below the water table will have normalized undrained shear strengths in a range of about 0.2 to 0.3 (e.g. Ladd et al. 1977). As indicated on Figure 5.2, the water table was estimated at 17 feet below the ground surface based on soil color change and water content data.

The most noteworthy observation from the subsurface investigation was the significant thickness of relatively compressible clay beneath this site. Oedometer results from four tests on representative samples from the clay layers are shown in Figure 5.4 and Table 5.2. Overconsolidation ratios (OCRs) shown in the table indicate the soil is generally lightly to moderately overconsolidated, consistent with normalized undrained shear strength mentioned above. Values of compression index (c_c) and recompression index (c_r) are generally lower than expected based on popular empirical correlations such as that proposed by Skempton (1944) and Kulhawy and Mayne (1990); however, results from two tests are consistent with values observed by one of the authors for several sites in Oklahoma (as shown in Figures 5.5 and 5.6). Interestingly, compression indices from the higher PI layer at a depth of 36 ft.

are similar in magnitude to the results obtained for the lower PI layer at a depth of 20 ft.

Settlement analysis based on the average results for parameters shown in Table 5.2 was conducted. Since oedometer data indicated relatively similar properties at different depths, a uniform clay layer with constant OCR was assumed. The embankment dimensions estimated from project plans and assumed stratigraphy are shown on Figure 5.7. Calculated total settlement based on average properties was 4.8 inches as shown in Figure 5.8, and consisted of 3.7 inches of virgin compression and 1.1 inches of recompression. Results of the settlement analysis shown in Figure 5.8 reveal that a significant amount (78%) of the total settlement is due to virgin compression while 22% is attributed to recompression in soil immediately beneath the embankment. This observation emphasizes the importance of having high quality estimates of settlement properties, especially preconsolidation stress, for soil immediately beneath the embankment where loading is the greatest. If a significant clay crust (weathered layer) had developed, expected settlements would be less. Using the range of settlement parameters in Table 5.2 gives a range of possible total settlements from 2.8 to 8.2 inches. Settlement predictions are consistent with the magnitude of observed settlements and are likely a primary cause of the approach slab settlement problem for SH-59A over Big Creek. In addition, as noted previously in this report, voids were observed under the slab indicating some erosion may also be contributing to the observed settlements.

5.2.2 Experimental Assessment of Ground Penetrating Radar for Detection of Voids Beneath Approach Slabs

As part of this project, researchers took the opportunity to assess how well a portable Ground Penetrating Radar (GPR) unit could detect voids beneath the approach slab. Dr. Jamie Rich of the University of Oklahoma Conoco Phillips School of Geology and Geophysics led this effort. The first attempt at employing this technology was made at the SH-59A over Big Creek site using a 400MHz antenna. The information and results presented below were obtained from Dr. Rich. To summarize:

- Collected GPR data in East bound lane, west side of bridge at 200MHz
    - 8 lines spaced 50cm apart, sample spacing 1.9”, saw very little evidence of voids.
- Collected GPR data in East bound lane, west side of bridge at 400MHz
    - 6 lines 50cm apart, sample spacing 1.9” evidence of voids on raw data.
- Collected GPR data in East bound lane, east side of bridge at 400MHz
    - 7 lines 50cm apart, 1st 4 lines sample spacing 4.7”, last 3 lines sample spacing 2.9”

In Figure 5.9 a photograph of the Portable GPR in use in is shown. Figure 5.10 presents the results along one line of the GPR survey and interpretation based on the 400MHz data. Based on this initial work the following conclusions were made:

- Presence of voids is apparent on 400MHz data.

- Perturbation of waveform over void can be used for detection and approximate extent, but poor resolution.
- A higher frequency antenna (1-1.6GHz) is recommended for better resolution and characterization of the voids.
    - A large contrast between concrete and air as reflected on the 400MHz waveform should give a distinct reflection on higher frequency data.

Following the initial work using the 400MHz antenna, a 1.3 GHz antenna was obtained and used at the same site for comparison. While data interpretation is continuing, the preliminary results reveal stronger reflections from discontinuities in the pavement and greater potential for detecting voids. Results and interpretation from the higher frequency antenna are shown Figure 5.11. These preliminary results indicate the GPR may be a valuable tool for void detection under slabs, as well as other uses.

Table 5.1 State Highway 59A over Big Creek Summary

State Highway 59 A over Big Creek		NBI:	24277	Constructed:	1996
County:	Pontotoc			*ODOT Division:*	3
Abutment Type:	Integral			*Traffic Direction:*	E & W
Site Description:					
Embankment Thickness:	7’	*Embankment Description:*	Clay and weathered shale		
Natural Deposit Thickness:	60’	*Natural Soil Description:*	Clay, sandy clay, and silty clay.		
Bedrock:	Shale				
Geology:	Pennsylvanian Age Pontotoc Group				
Site Condition:					
Driving Surface:	Rough asphalt Moderate bump	*Likely issues:*	Foundation settlement, erosion & drainage		
Abutment:	Leaching				
Under Approach:	Voids				
Maintenance:	Asphalt	*ADT (2008)*	1,100		

Bridge information obtained from ODOT Design Sheets, Bridge Inspection Reports, GRIP Lite, ODOT Geology Manuals, and Geology Map of Oklahoma

Table 5.2 Summary of Oedometer Results for SH-59A over Big Creek

Boring	Test Type	Depth (ft)	Elev. (ft)	P'c (tsf)	P'o (tsf)	OCR	Cc	Cr	eo
BH-1	In Situ	20.1	962.7	2.61	1.03	2.5	0.275	0.018	0.83
BH-1	Soaked	20.8	962.0	1.46	1.04	1.4	0.238	0.028	0.69
BH-2	In Situ	36.0	946.8	3.24	1.47	2.2	0.287	0.027	0.88
BH-2	Soaked	36.0	946.8	2.19	1.47	1.5	0.278	0.014	0.82

Notes: “In situ” implies samples were tested at natural water content without adding water. “Soaked” implies samples were fully submerged during the test.

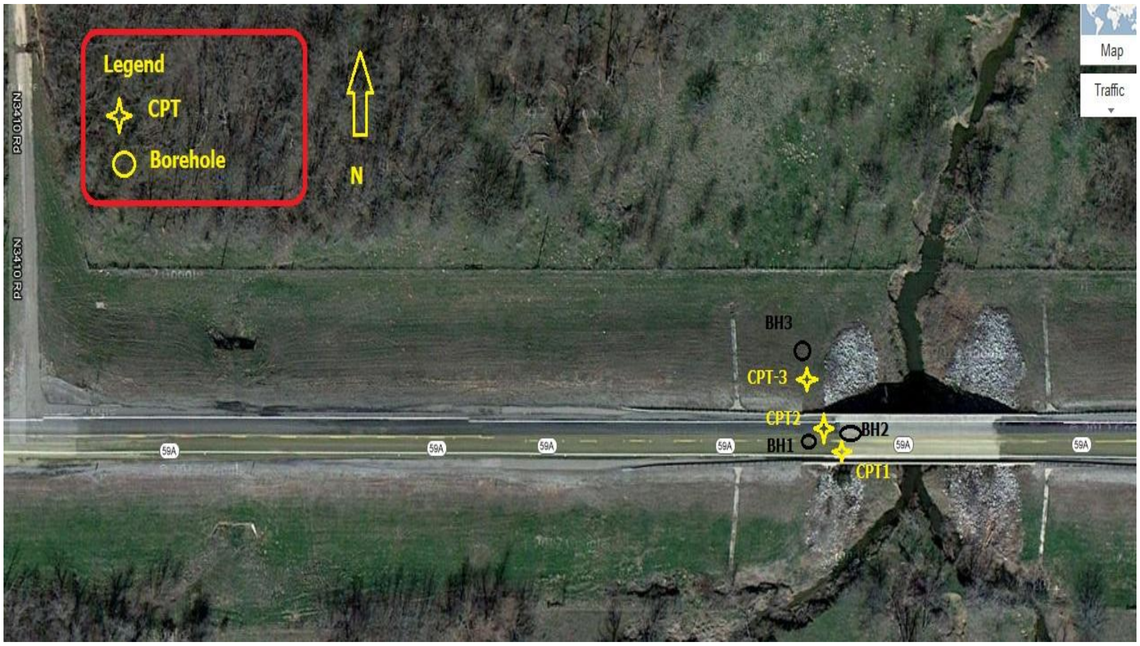

Figure 5.1 Sounding Locations for SH-59A over Big Creek

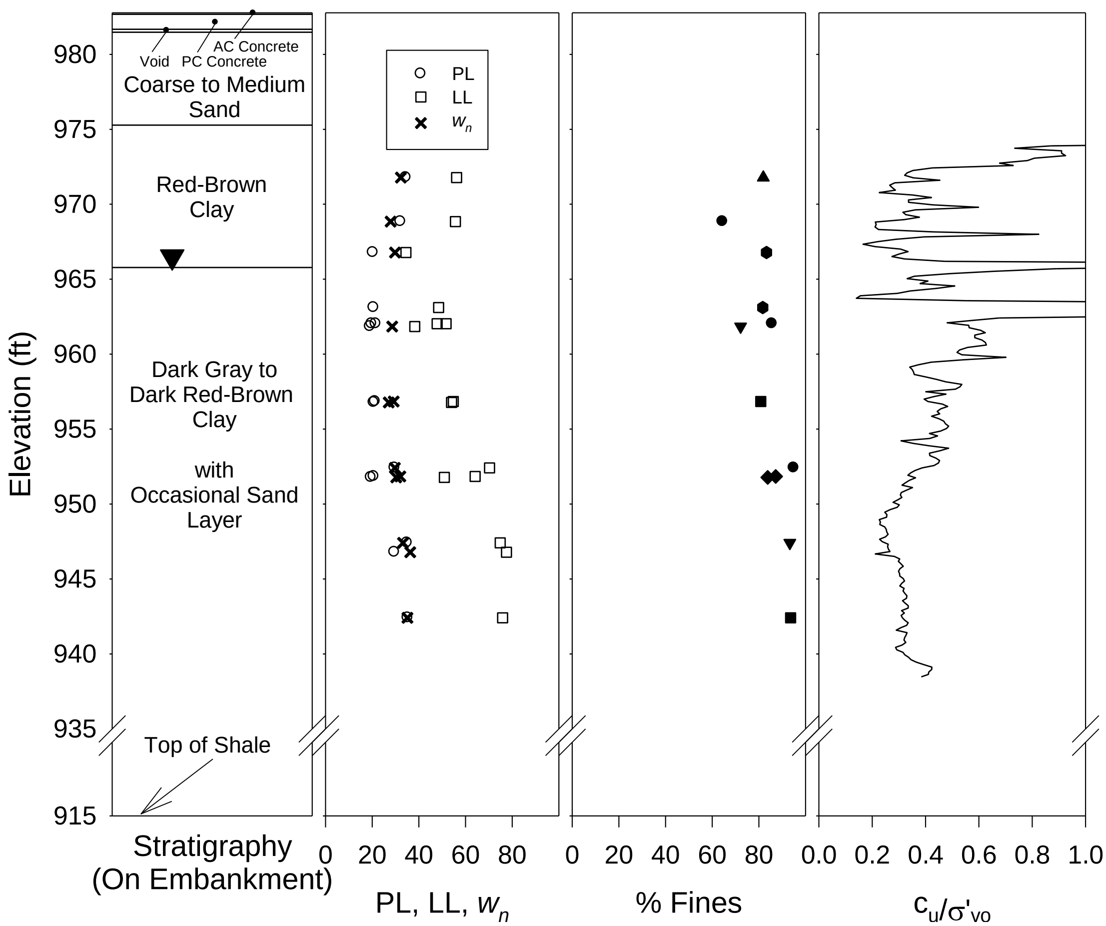

Figure 5.2 Soil Properties for West Embankment – SH-59A over Big Creek

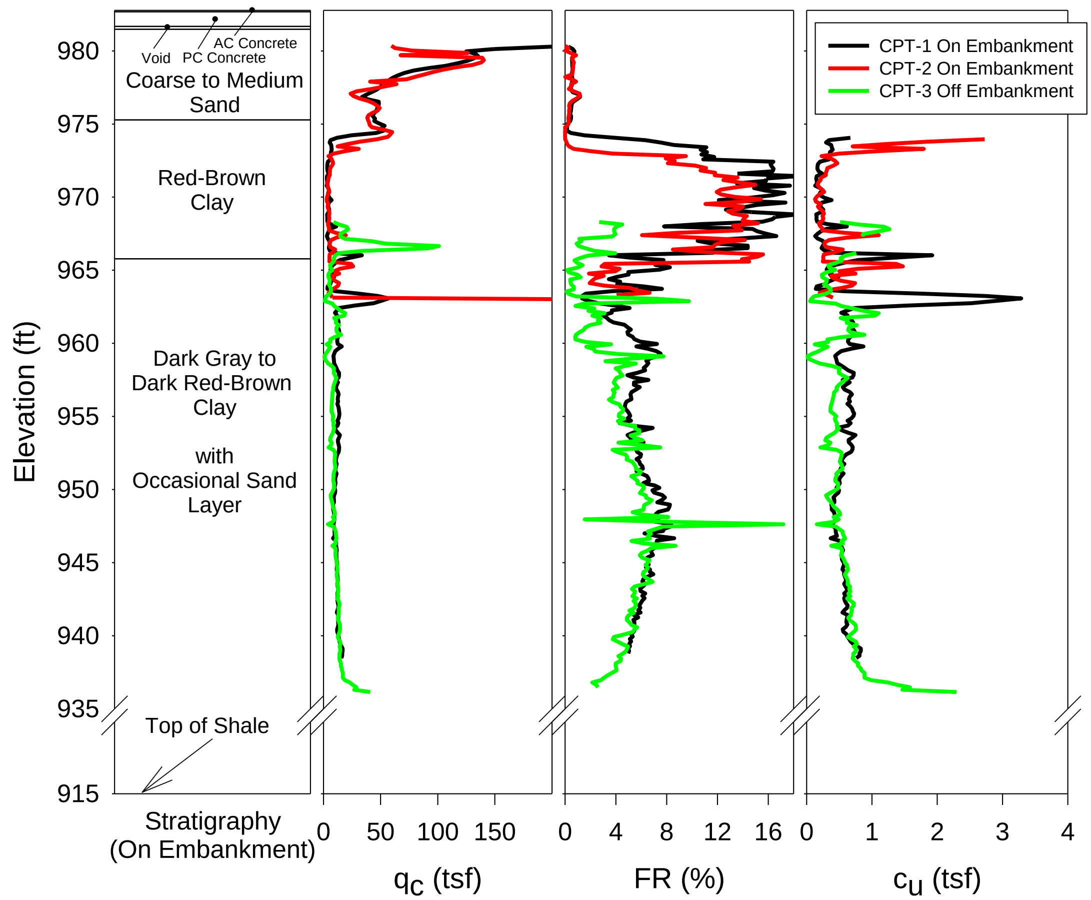

Figure 5.3 Cone Soundings for West Embankment – SH-59A over Big Creek

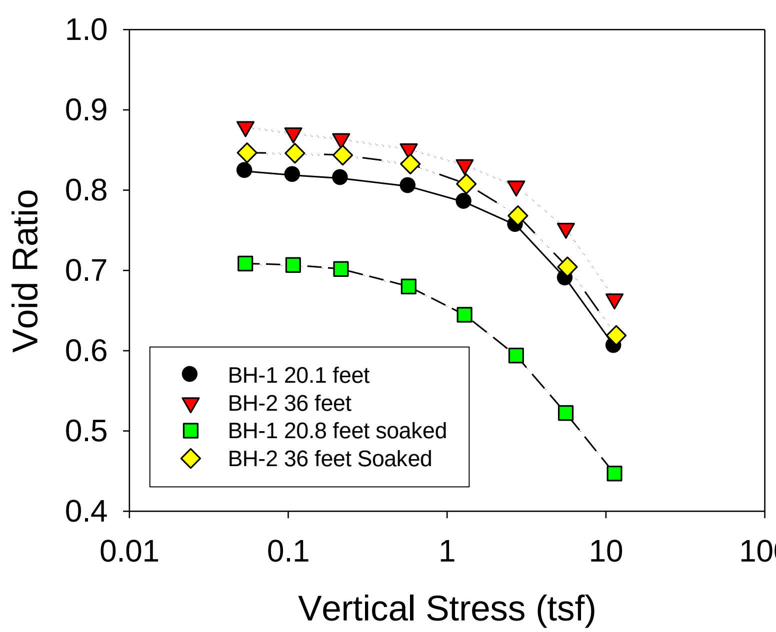

Figure 5.4 Oedometer Test Results - SH-59A over Big Creek

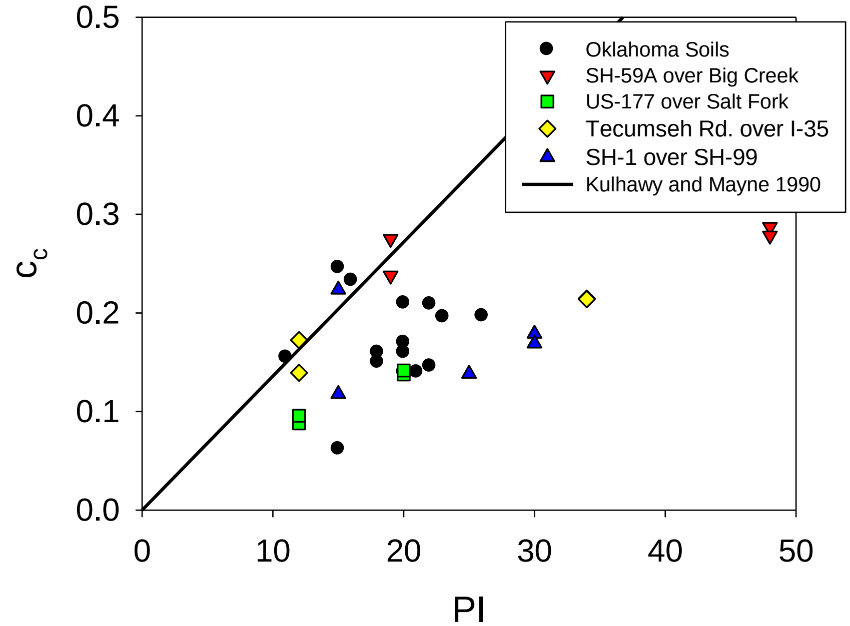

Figure 5.5 Virgin Compression Index versus PI for Oklahoma Soils

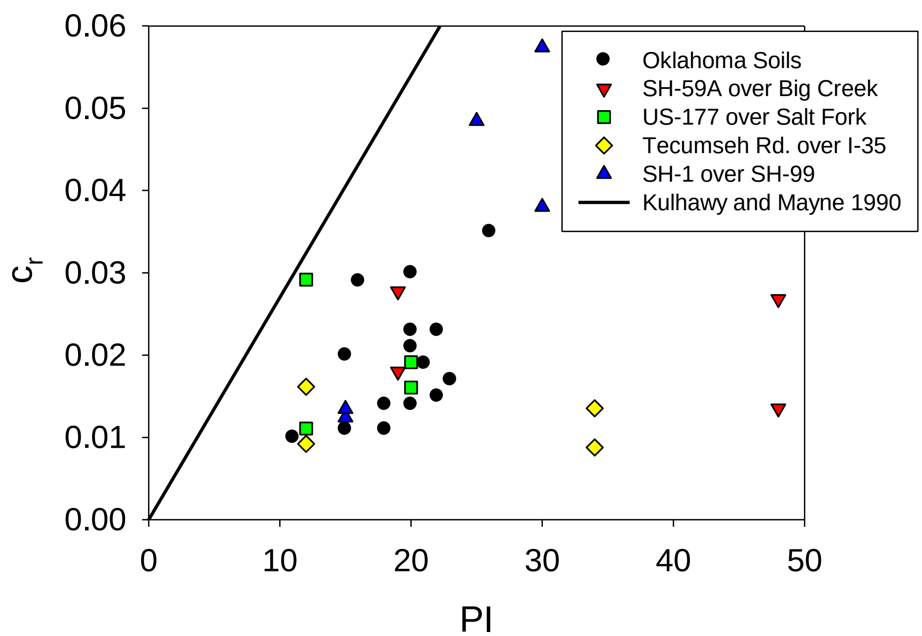

Figure 5.6 Recompression Index versus PI for Oklahoma Soils

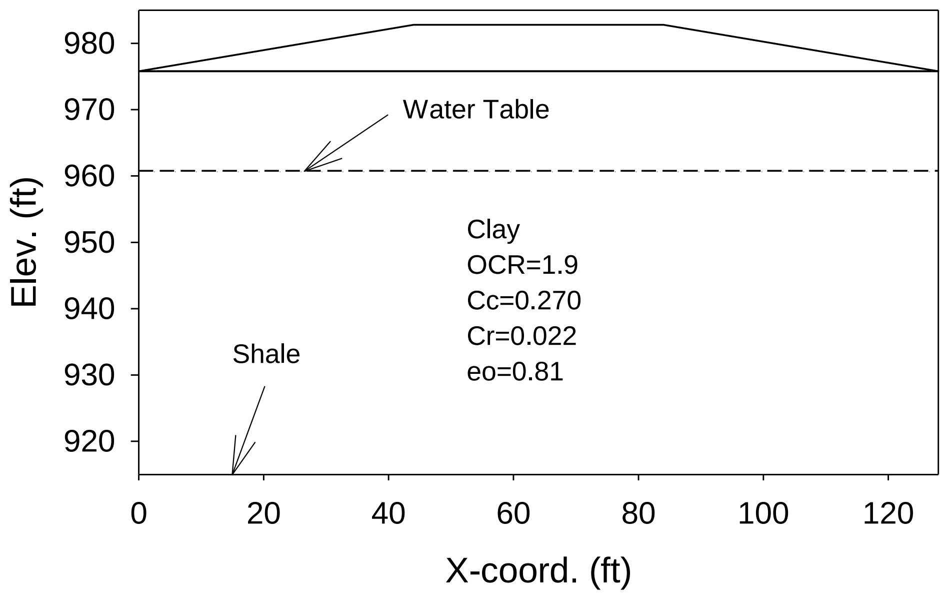

Figure 5.7 Embankment Geometry and Soil Profile Assumed for Settlement Analysis - SH-59A over Big Creek

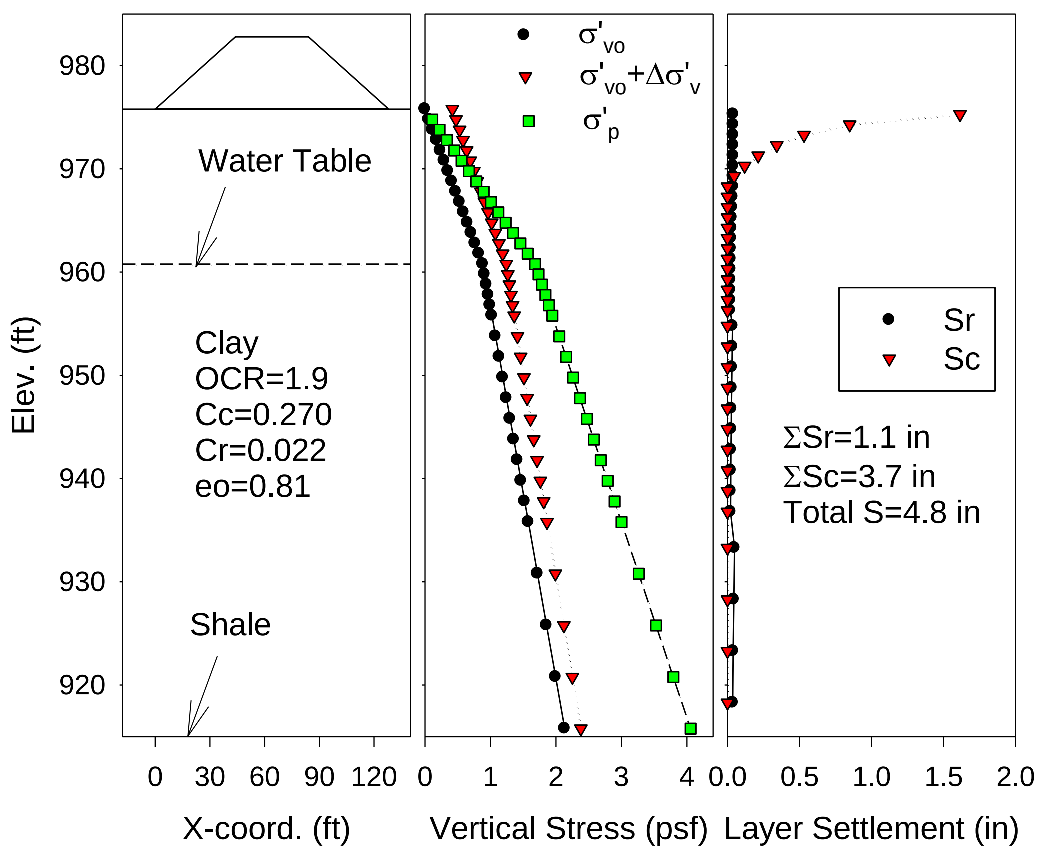

Figure 5.8 Results of Settlement Analysis - SH-59A over Big Creek

Figure 5.9 Photograph Showing Use of Portable GPR at SH-59A over Big Creek

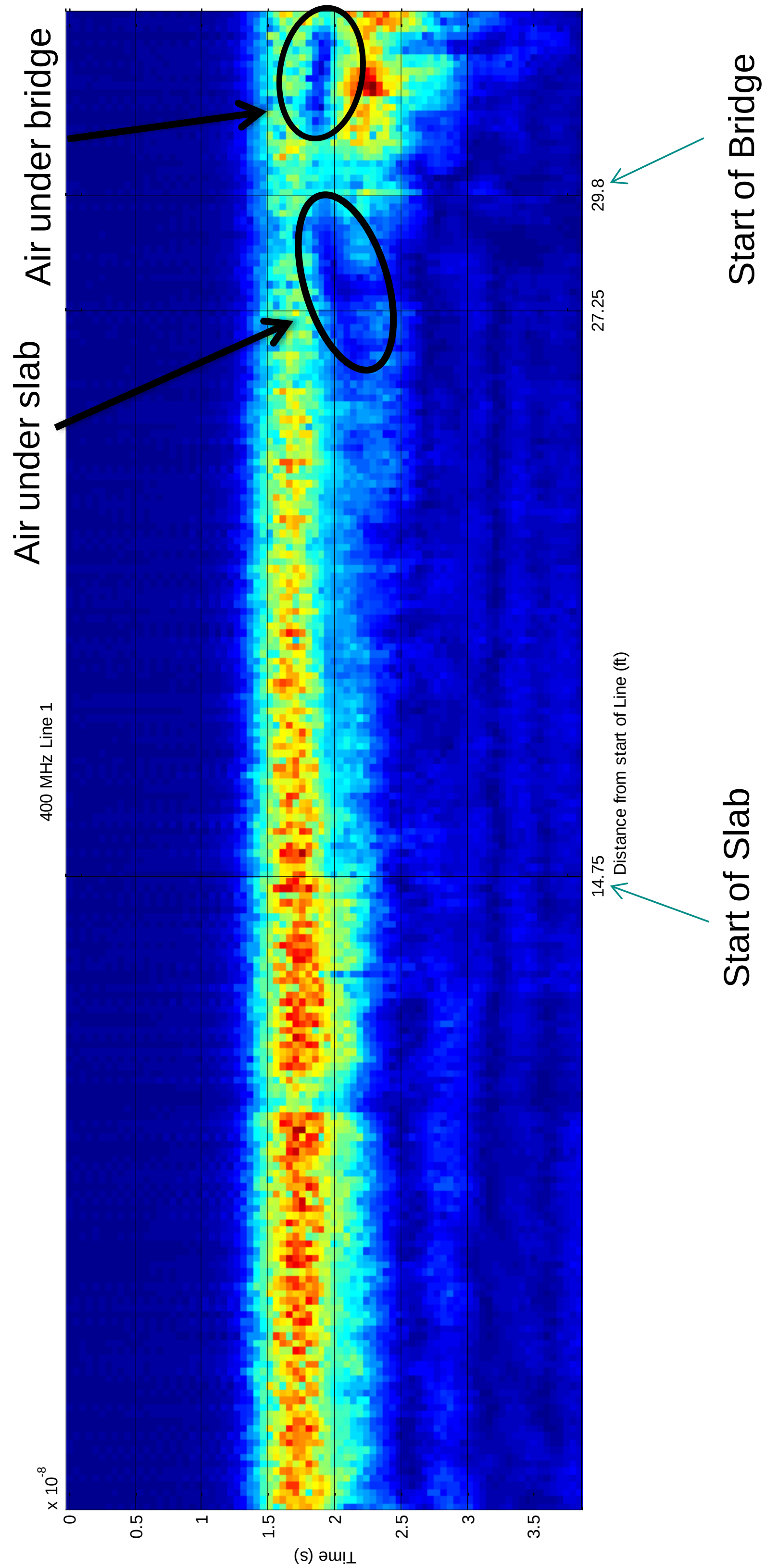

Figure 5.10 Results and Interpretation of GPR Along a Line on the West Approach Slab in the Eastbound Lane – 400MHz Antenna

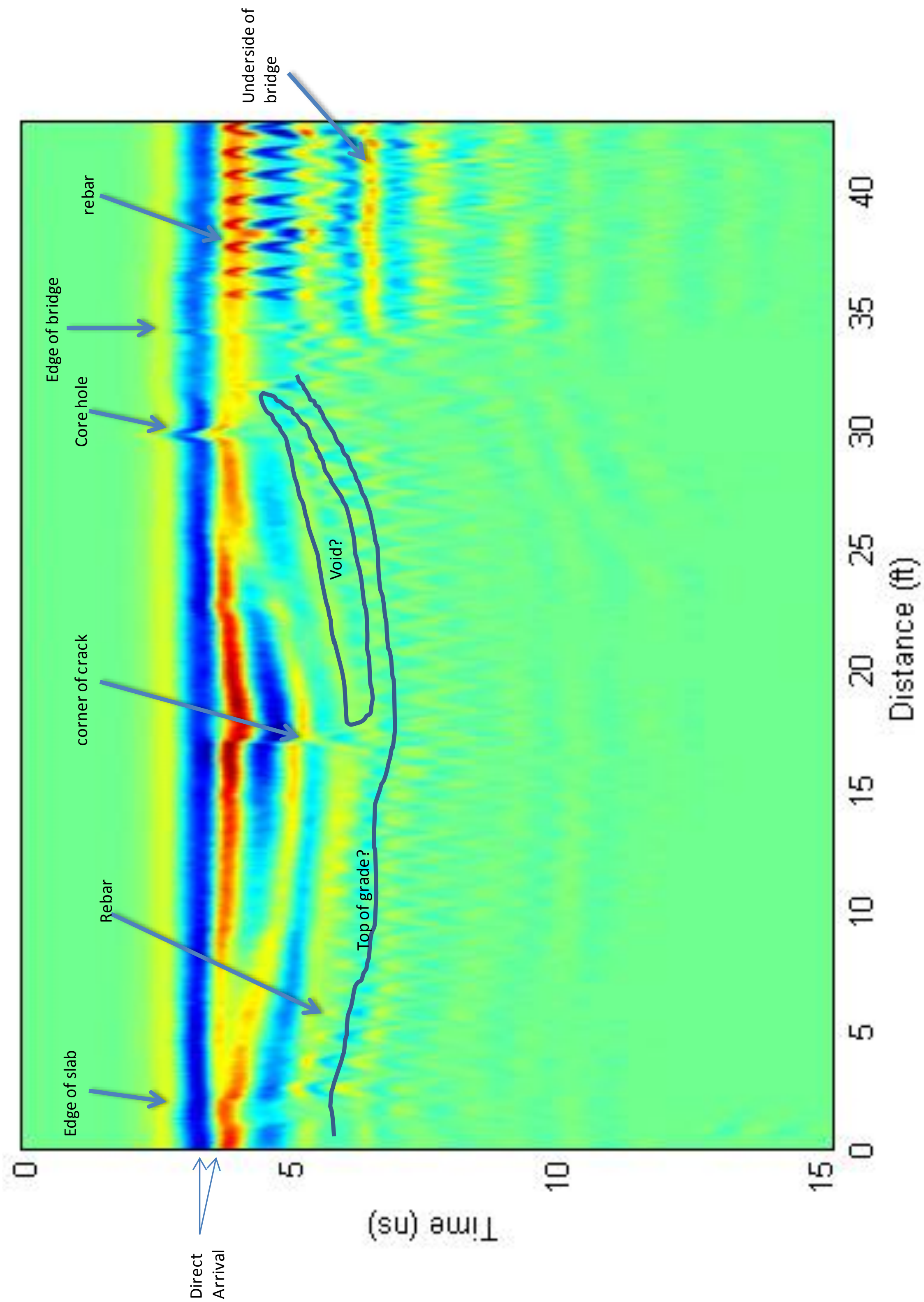

Figure 5.11 Results and Interpretation of GPR Along a Line on the East Approach Slab in the Westbound Lane – 1.3 GHz Antenna

5.3 US-177 OVER SALT FORK BRIDGE A

US-177 over Salt Fork Bridge A is a fairly long bridge with evidence of various issues related to the observed approach slab settlement. Erosion channels running under the abutment were observed and abutment piles are exposed. Bumps have developed at the bridge-approach interface. Multiple asphalt overlays appear to have been placed to smooth the transition but these too have settled and cracked.

This bridge was selected based on the embankment size, the foundation soil, and extent of the problem. This site was the subject of an extensive study investigating the performance of abutment backfill design and construction methods (Snethen et al. 1997). Bridge A was the control in a three bridge study. The embankment backfills were constructed using typical methods; unclassified borrow material was used. At the time of that study, a bump was developing at approach A2 and the approach was not performing as well as some of the experimental approaches (Snethen et al. 1997). Table 5.3 presents a summary of information on this bridge.

5.3.1 Results of Subsurface Investigation

Seven test borings and eight cone soundings were conducted, both on and off the north and south embankments of Bridge A, as shown in Figures 5.12 and 5.13. Test boring logs are contained in the appendix. Soil properties and interpreted soil profiles are shown in Figure 5.14 for the south embankment, and results of cone soundings are shown in Figures 5.15 and 5.16. Since soils below the north embankment were primarily sand, the focus of the settlement analysis was on the south embankment where significant clay layers were detected. In addition, the south embankment was about twice as high as the north embankment. Test borings for the south embankment reveal about 8 feet of granular backfill, mostly fine sand behind the abutments underlain by about 24 feet of red to dark brown sandy clay that primarily makes up the embankment fill. Plasticity indices in the clayey fill range from 12 to 24 with an average around 18, and sand contents range from 61% to 86%. Cone data in Figure 5.15 reveal the soil below the fill varies from sand to clay at the south embankment. The cone data from the CPT-6 test on the west side of the embankment (off the embankment) reveal a strong layer of sand in the soils at elevations below the embankment fill. On the other hand, CPT-7 on the east side of the embankment (off the embankment) reveals a soft layer of clayey soil for the same elevations. This is of particular concern since soft clays are highly compressible and can settle significantly. Below this variable zone, both CPT-6 and CPT-7 indicate a 6-foot layer of sand followed by about 10 feet of stiff clay to the termination depth.

Normalized undrained shear strengths (c_u/σ'_{vo}) interpreted from CPT results, shown in Figure 5.14, generally indicate the clayey soils that makeup the embankment exhibit moderately overconsolidated behavior, with occasional highly overconsolidated stiffer layers. The soft clay layer detected by CPT-7 exhibits normalized strengths consistent with normally consolidated clay. As indicated on Figure 5.14, the water table was estimated at 42 feet below the top (Elev. 932) of the embankment based on test boring logs.

Two noteworthy observations from the subsurface investigation were the variability in the alluvial stratigraphy (CPT-6 vs. CPT-7) and the possible presence of a soft compressible clay layer below the fill (CPT-7). Settlement analysis considering only compression of the possible soft clay layer of 10-foot thickness was conducted based on oedometer results shown in Figure 5.17, and Table 5.4, and layer geometry shown in Figure 5.18. This layer was detected by the CPT testing; however, tube samples were not obtained from this soil since it was not encountered during drilling and sampling. Therefore, compression indices shown in Figure 5.19 are assumed values similar to those shown in Table 5.4. These are consistent with other Oklahoma soils as shown in Figures 5.5 and 5.6. Assuming the soft layer to be normally consolidated, predicted settlement using a modest virgin compression index of 0.15 is about 9 inches. It seems likely that compression of softer material beneath portions of the embankments has contributed to the observed settlement problem.

In addition to the settlement of foundation layers, compression of the fill material may have contributed to the overall settlement of the bridge approach slabs. Figure 5.20 shows double-oedometer data for a composite sample of embankment fill soil taken from 15 to 18 feet in Boring BH-1. Two samples were compacted to a relative compaction of approximately 97% and a moisture content 2 percentage points dry of optimum (approximate w=13.2%) based on the standard Proctor curve shown in Figure 5.21. This soil state is meant to represent the average compacted condition dry of optimum but within specifications. In Figure 5.20, the soil exhibits a significant tendency for swelling up to a stress of about 0.91 tsf corresponding to the point at which the soaked and as-compacted curves cross. This is the equivalent of about 14 feet overburden pressure. Soil above a depth of 14 feet may exhibit swelling while soil below a depth of 14 feet may exhibit collapse settlement upon wetting. For this embankment, the zone subject to wetting-induced collapse would be from a depth of 14 feet to the embankment bottom at 30 feet. Predicted collapse settlement is about 2.3 inches for complete wetting of the soil zone below a depth of 14 feet. In Figure 5.14, natural moisture contents in the fill are near to or slightly above the plastic limit and corresponding degrees of saturation calculated from Shelby tube sample measurements vary between 87% and 100% with an average of about 95%. Thus, while some wetting appears to have occurred since construction, on average the embankment soil has not experienced complete wetting. It follows that some small amount of the observed approach slab settlement may be due to wetting-induced collapse; however, it seems in this case other factors are greater contributors to the problem.

Table 5.3 US-177 over Salt Fork Summary

U.S. 177 over Salt Fork		**NBI:**	**24475**	**Constructed:**	**1997**
County:	Noble			*ODOT Division:*	4
Abutment Type:	Non-Integral			*Traffic Direction:*	N & S
Site Description:					
Embankment Thickness:	North: 15' South: 30'	*Embankment Description:*	Unclassified Borrow Compacted Fill		
Natural Deposit Thickness:	North: 30' South: 15'	*Natural Soil Description:*	*North*: Lean clay near surface, sandy soils deeper.		
Bedrock:	Shale		*South*: Predominately fat clay, some lean clay		
Geology:	Permian Age Wellington Unit				
Site Condition:					
Driving Surface:	Moderate bump	*Likely issues:*	Embankment settlement, foundation settlement, and erosion issues		
Abutment:	Voids				
Under Approach:					
Maintenance:	Asphalt overlay, which has also settled.	*ADT (2008)*	3,900		

Bridge information obtained from ODOT Design Sheets, Bridge Inspection Reports, GRIP Lite, ODOT Geology Manuals, and Geology Map of Oklahoma

Table 5.4 Summary of Oedometer Results for US-177 over Salt Fork

Boring	**Test Type**	**Depth (ft)**	**Elev. (ft)**	**P'c (tsf)**	**P'o (tsf)**	**OCR**	**Cc**	**Cr**	**eo**
BH-1	In Situ	19.8	912.3	2.61	1.98	1.3	0.138	0.016	0.52
BH-1	Soaked	19.4	912.7	2.61	1.97	1.3	0.142	0.019	0.57
BH-3	In Situ	16.7	915.4	2.82	1.85	1.5	0.088	0.029	0.43
BH-3	Soaked	16.0	916.1	1.67	1.85	0.9	0.096	0.011	0.42

Notes: “In situ” implies samples were tested at natural water content without adding water. “Soaked” implies samples were fully submerged during the test.

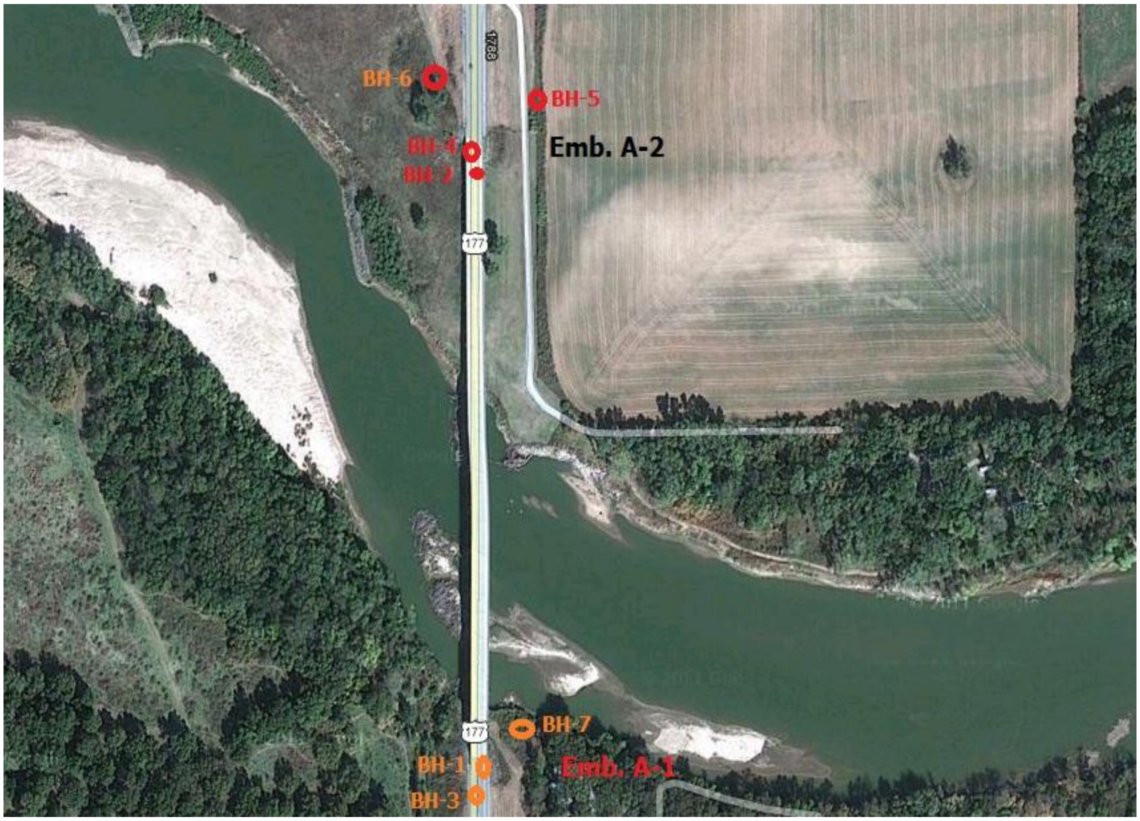

Figure 5.12 Borehole Locations for US-177 over Salt Fork

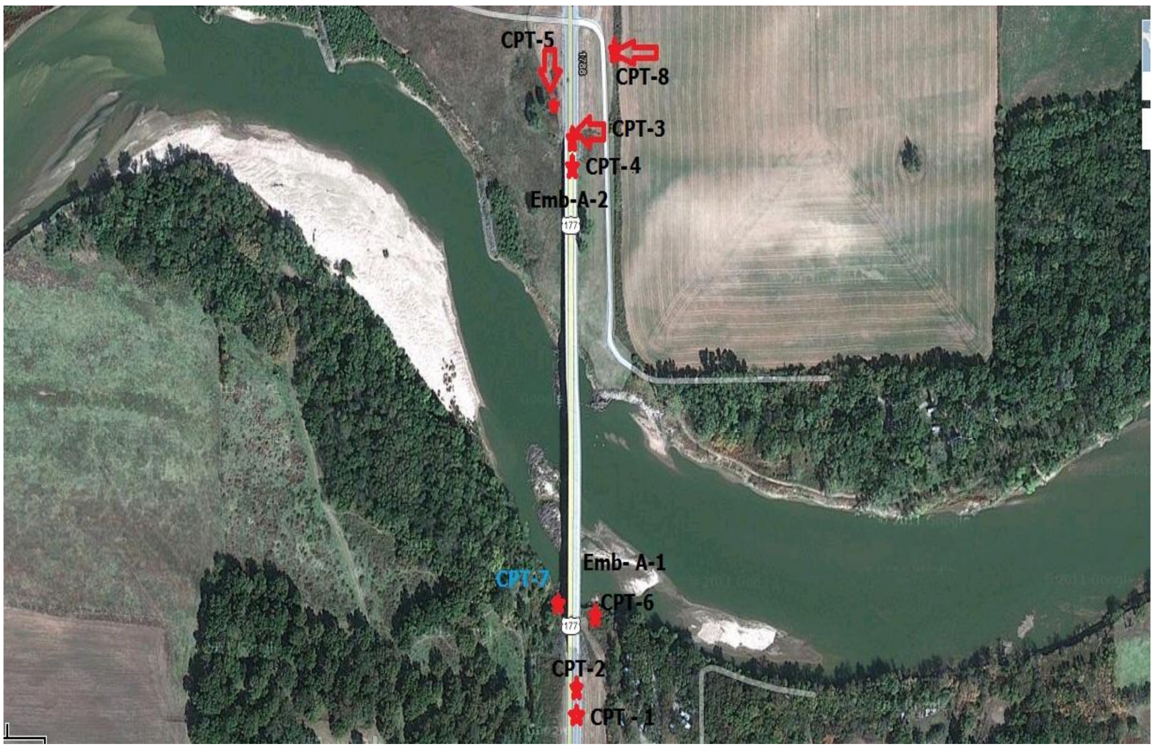

Figure 5.13 CPT Locations for US-177 over Salt Fork

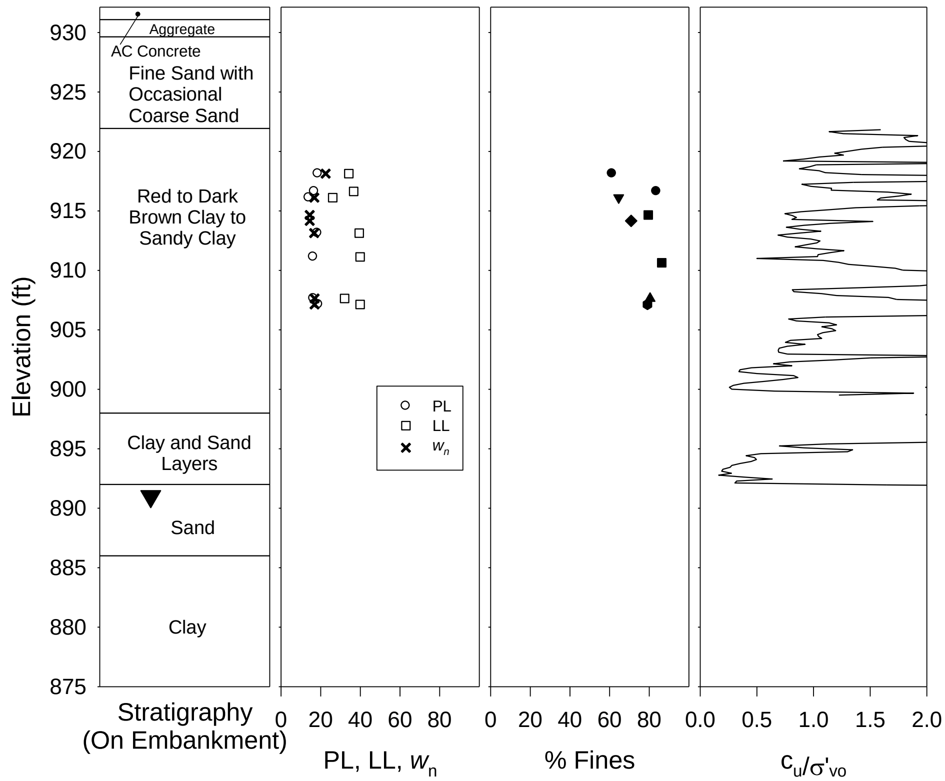

Figure 5.14 Soil Properties for South Embankment – US-177 over Salt Fork

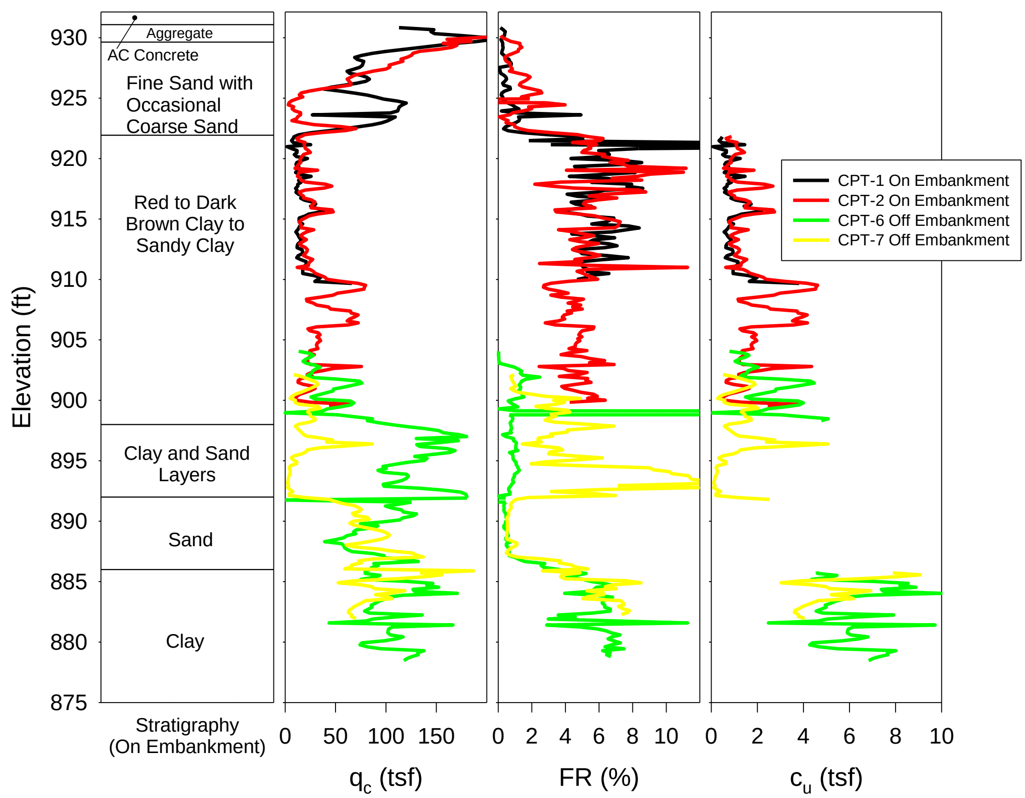

Figure 5.15 Cone Soundings for South Embankment – US-177 over Salt Fork

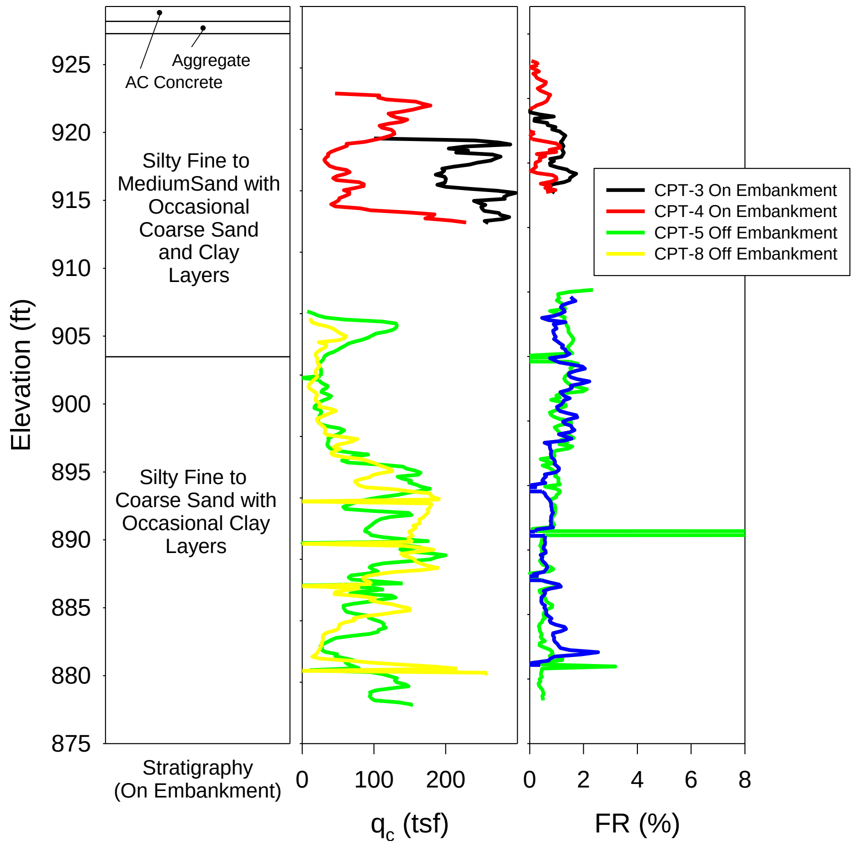

Figure 5.16 Cone Soundings for North Embankment – US-177 over Salt Fork

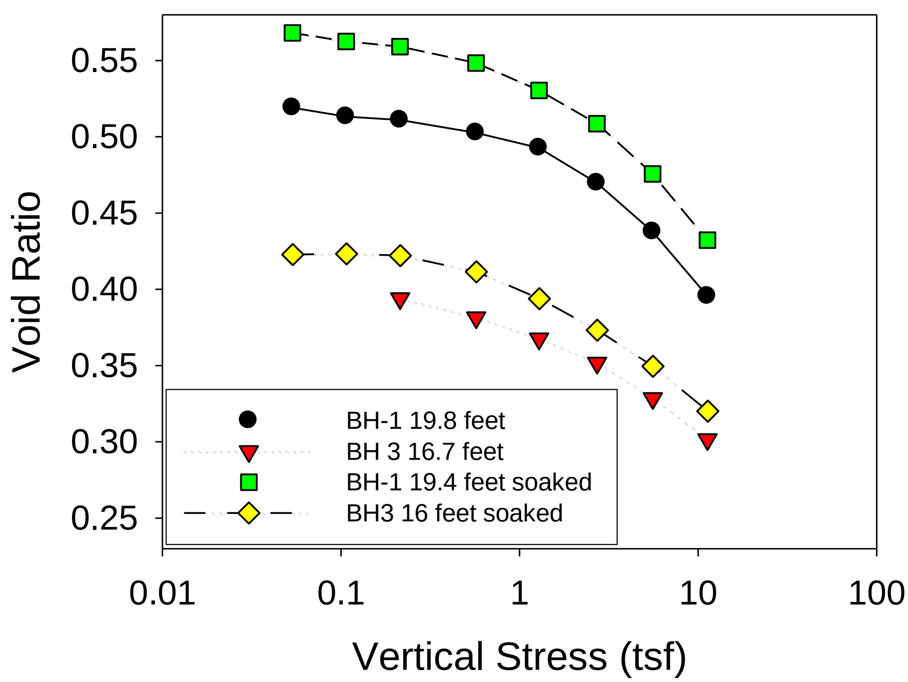

Figure 5.17 Oedometer Test Results – US-177 over Salt Fork

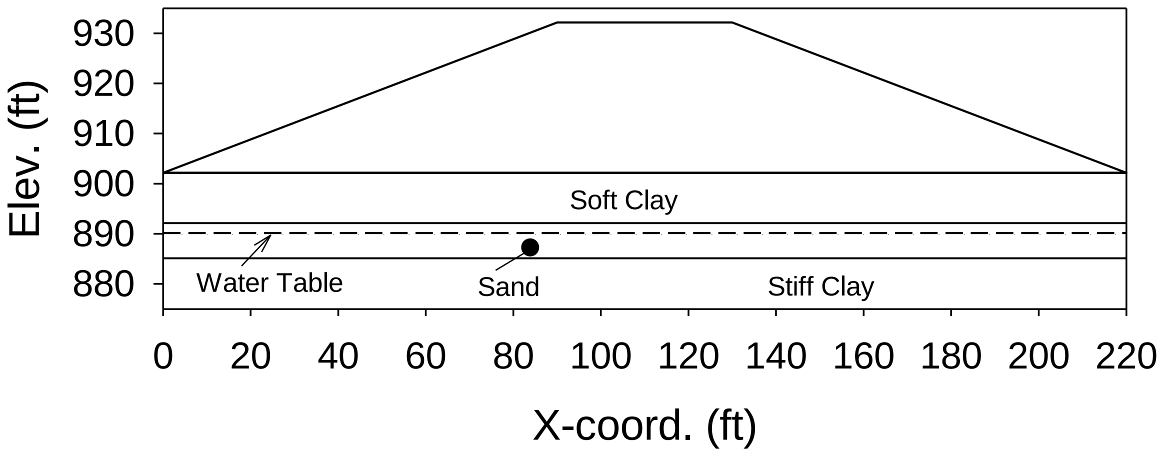

Figure 5.18 Embankment Geometry and Soil Profile Assumed for Settlement Analysis – US-177 over Salt Fork

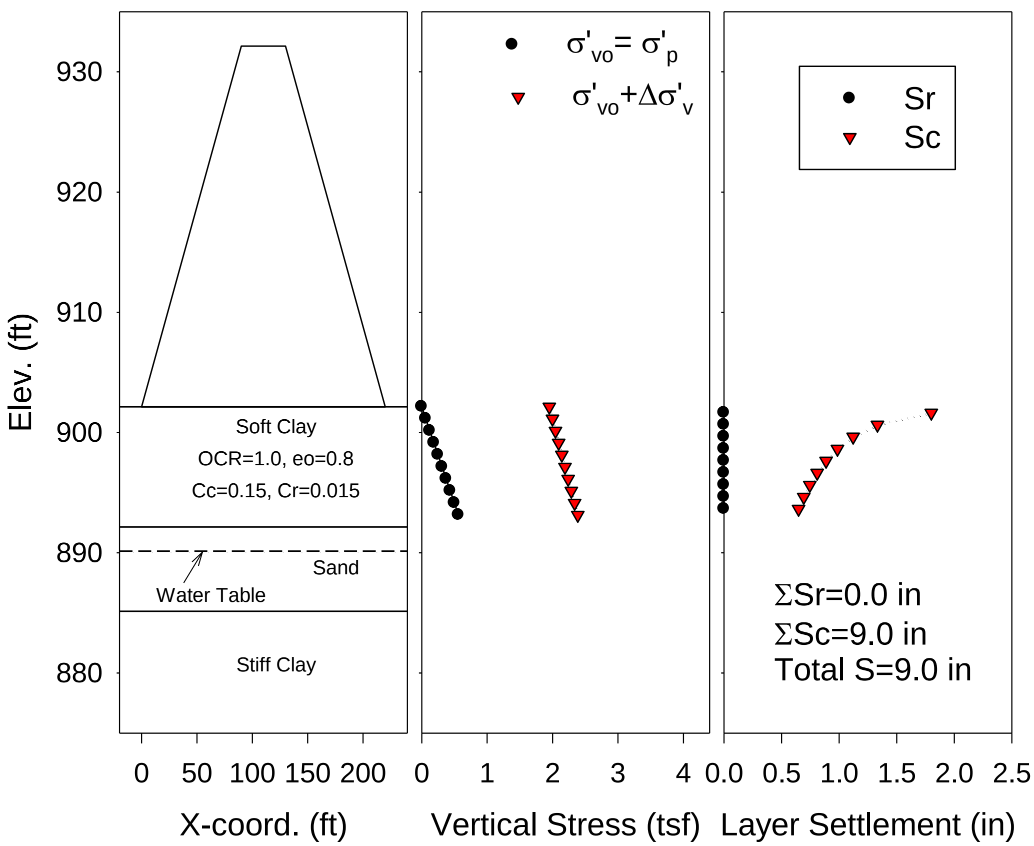

Figure 5.19 Results of Settlement Analysis – US-177 over Salt Fork, South Embankment

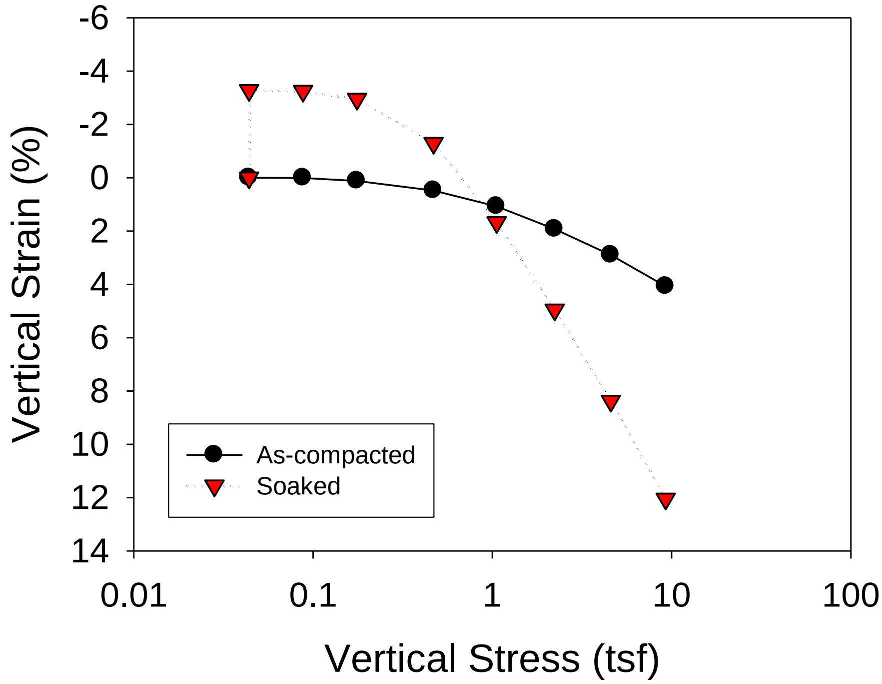

Figure 5.20 Double-Oedometer Results – US-177 over Salt Fork: BH-1, 13-18 feet

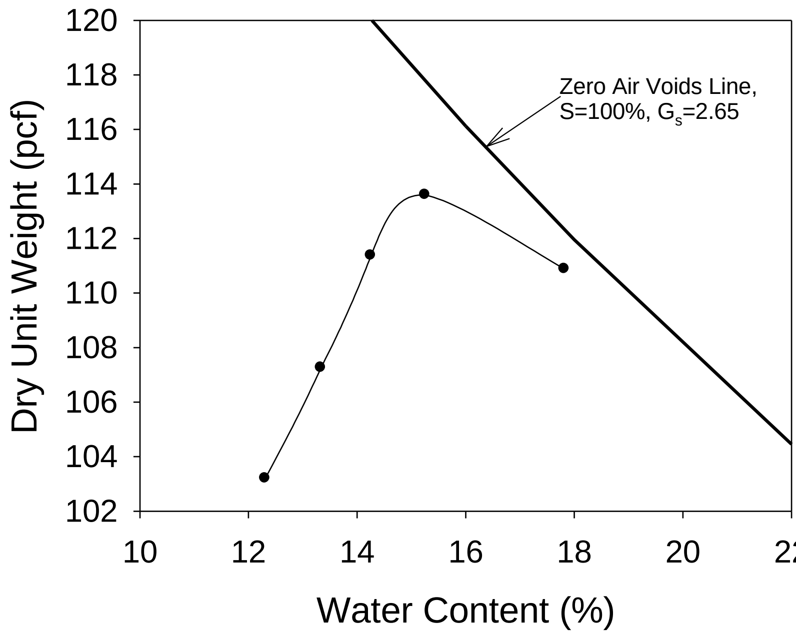

Figure 5.21 Standard Compaction Curve - US-177 over Salt Fork: BH-1, 13-18 feet

5.4 STATE HIGHWAY 6 OVER WEST ELK CREEK

This bridge is an integral abutment bridge that carries both directions of State Highway 6 over West Elk Creek in Elk City. Bridge approach settlement was first recorded in maintenance records in 2007. The approaches consist of two slabs that have settled in a “V” shape; the road and bridge ends are higher than where the two approach slabs meet. There are significant voids beneath the approach slab and abutments and severe bumps have developed at each end of the bridge.

This site was chosen because the severe bump on the driving surface and the voids present beneath the structure. There is very little fill, so the behavior of foundation soil can be fairly well isolated in this case. The borings do not indicate the presence of cohesive soils, so time consolidation should not be an issue at this site. Table 5.5 presents a summary of information about this bridge.

5.4.1 Results of Subsurface Investigation

Four test borings and five cone soundings were conducted, both on and off the north and south embankments, as shown in Figure 5.22. Test boring logs are contained in the appendix. Results of cone soundings and interpreted soil profiles are shown in Figures 5.23 and 5.24.

The soil profile on both ends of the bridge appears similar. It consists of about one foot of pavement underlain by 9 feet of coarse to medium sand backfill, below which is a reddish-brown silty sand with occasional thin clay seams and weathered sandstone fragments extending to bedrock about 29 feet below the top of the pavement.

It seems unlikely that settlement of the foundation soil or wetting-induced volume change in the embankment soil is the cause of approach slab settlement in this case. The granular backfill behind the abutment appears relatively dense based on the cone tip resistance, foundation soils are primarily cohesionless silty sand and embankment heights are relatively small. If settlements did occur, they should have occurred during construction given the permeable granular soils and relatively deep water table. On the other hand, as noted in Chapter 4, extensive erosion under the abutments and approach slabs was noted during field visits and voids were detected beneath the pavement during test borings. It appears in this case, undermining of the approach slabs by erosion may be the primary cause of the problem.

Table 5.5 State Highway 6 over West Elk Creek Summary

State Highway 6 over West Elk Creek		**NBI: 25128**	**Constructed: 1999**
County:	Beckham	*ODOT Division:*	5
Abutment Type:	Integral	*Traffic Direction:*	N & S
Site Description:			
Embankment Thickness:	1-2’	*Embankment Description:*	Silty sand and crushed sandstone
Natural Deposit Thickness:	5-7’	*Natural Soil Description:*	Silty sand
Bedrock:	Shaley Sandstone Sandstone		
Geology:	Pennsylvanian Age Shale, Pontotoc Group		
Site Condition:			
Driving Surface:	Severe bump	*Likely issues:*	Erosion, potentially foundation settlement
Abutment:	Moderate voids		
Under Approach:	Mild voids		
Maintenance:		*ADT (2008)*	4,900

Bridge information obtained from ODOT Design Sheets, Bridge Inspection Reports, GRIP Lite, ODOT Geology Manuals, and Geology Map of Oklahoma

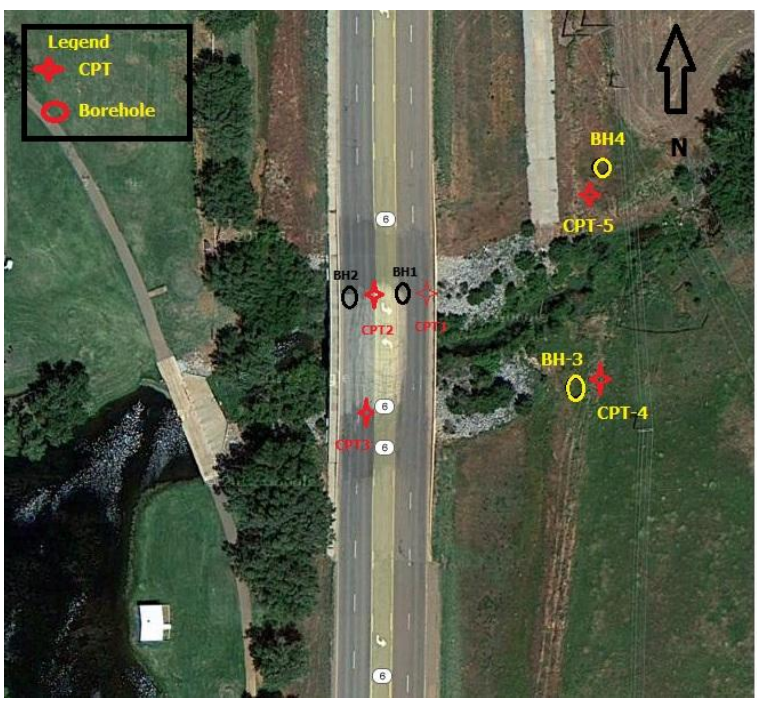

Figure 5.22 Sounding Locations for SH-6 over West Elk Creek

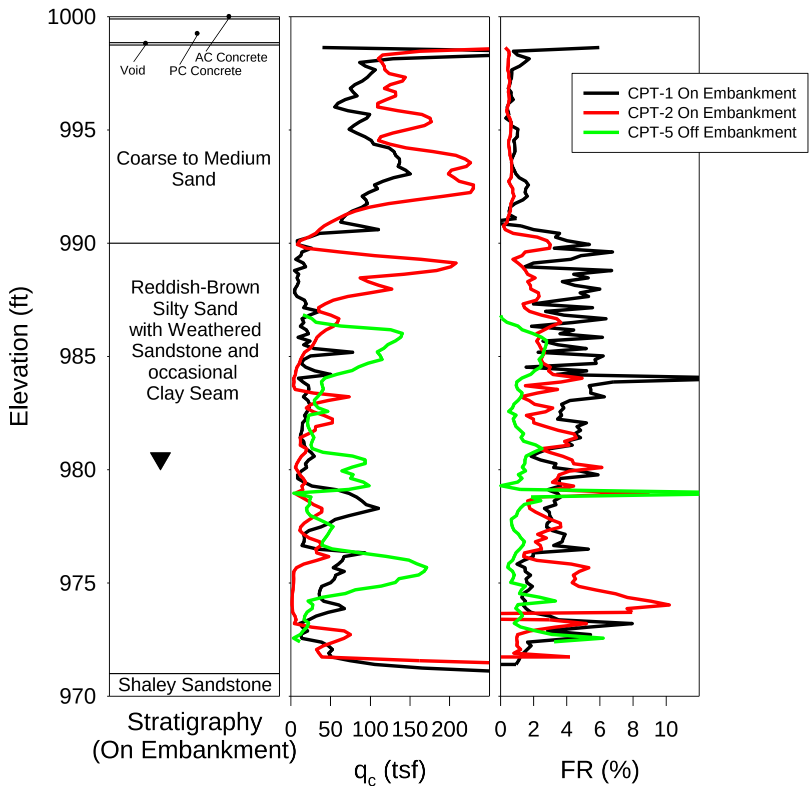

Figure 5.23 Cone Soundings for North Embankment – SH-6 over West Elk Creek

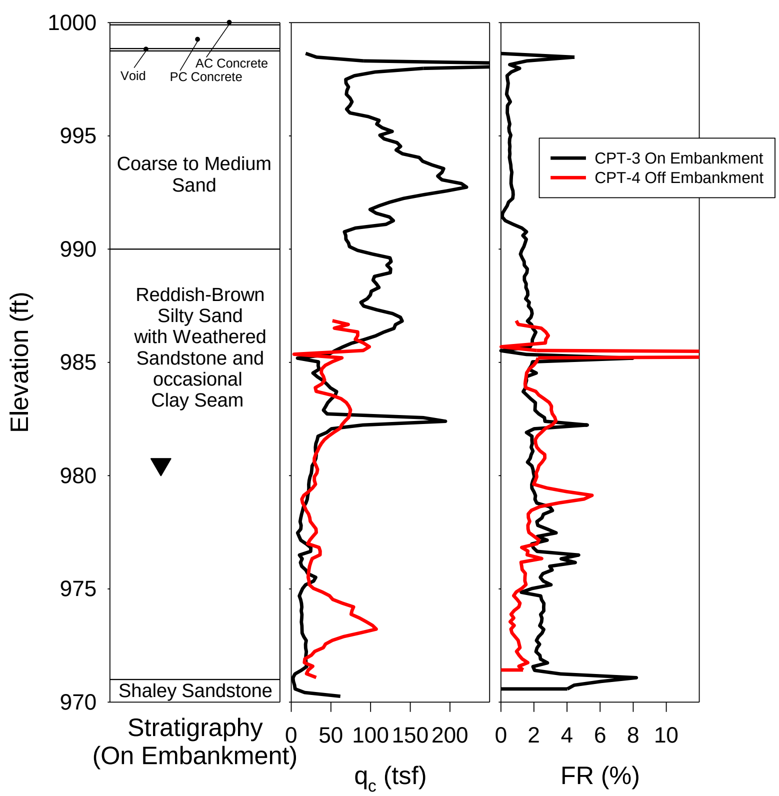

Figure 5.24 Cone Soundings for South Embankment – SH-6 over West Elk Creek

5.5 TECUMSEH ROAD OVER INTERSTATE 35

This conventional abutment bridge was constructed in 1997 and carries substantial east and westbound traffic over Interstate 35. Settlement of the approach slabs was noted on a bridge inspection made in April 2001, and problems continued to worsen in subsequent reports. There is evidence of horizontal displacement of the wing walls away from the bridge. Uretek® foam has been pumped under the slabs; however, voids can be observed beneath the foam, indicating that the problem continued to develop after maintenance was performed. Evidence of erosion was observed below the slope wall.

This site was selected based on the severity and continuing settlement of the approach slab. The embankment height and foundation soil provide soil settlement potential and the evidence of erosion is also present. Table 5.6 presents a summary of this bridge.

5.5.1 Results of Subsurface Investigation

Five test borings and seven cone soundings were conducted, both on and off the west and east embankments, as shown in Figure 5.25. Test boring logs are contained in the appendix. Soil properties and interpreted soil profiles are shown in Figure 5.26 for both embankments. Since cone profiles are similar on both sides of the bridge, as shown in Figures 5.27 and 5.28, soil properties from both sides are combined in Figure 5.26. As shown in Figures 5.26, 5.27, and 5.28, soundings near the abutment encountered medium sand backfill to a depth of about 15 feet below the top of the pavement while soundings further back from the abutment encountered a red clayey fill over the same depths.

Below the upper 15 feet, approximately 11 feet of red clayey soil was encountered, followed by 9 feet of dark green-gray clay above shale. The red clayey soil contains on average 75% fines with a range of about 53% to 90%. Plasticity indices in the red clayey soil range from 12 to 28 with an average around 18. Natural water contents are generally low in the soil profile, below or near the plastic limit of the soil – this may in part be due to the prolonged drought conditions prior to the subsurface exploration.

Normalized undrained shear strengths (c_u/σ'_{vo}) interpreted from CPT results, shown in Figure 5.26, generally indicate the clayey soils that make up the upper 15 feet of the embankment exhibit moderately to highly overconsolidated behavior. However, the 20 feet of red clay below this exhibits normalized undrained strengths consistent with normally consolidated to moderately overconsolidated clay. Possibly the presence of softer layers may have contributed to the approach slab settlement. As indicated on Figure 5.26, the water table was estimated at 26 feet below the top of the embankment based on color changes and observations noted on test boring logs.

Looking at the soil profile, it is possible that consolidation of the green-gray clay layer due to the weight of the embankment may have contributed to the approach slab settlement. Oedometer results are summarized in Table 5.7. As shown in Figures 5.5 and 5.6, compression indices used in the settlement analysis are consistent with other Oklahoma soils. These consolidation parameters are based on the results of oedometer tests on samples obtained below the elevation of the bottom of the embankment. Using the idealized geometry shown in Figure 5.30 and the soil settlement properties shown in Figure 5.31, results in an estimated consolidation settlement of about 10.7 inches from compression of the clay layer. While actual settlements of the slab may be less, this analysis suggests that consolidation settlement of the natural clay soil beneath the embankment is a likely contributor to observed approach slab settlement. Some of this settlement would have likely occurred during construction and so the full amount of settlement of the completed slab due to this mechanism was likely less than predicted. However, other mechanisms such as: erosion of soil observed under the slab, settlement due to lateral movement of wing walls, and wetting-induced volume change due to post-construction wetting are also likely contributors. The former two problems have been discussed previously in this report.

Regarding wetting-induced compression, Figure 5.32 shows double-oedometer data for a composite sample of embankment fill soil taken from 6 to 14 feet using a hand auger. Two samples were compacted to a relative compaction of approximately 97% and a moisture content 2 percentage points dry of optimum (approximate w=13.0%) based on the standard Proctor curve shown in Figure 5.33. This soil state is meant to represent the average compacted condition dry of optimum but within specifications. In Figure 5.32, the soil exhibits a significant tendency for swelling up to a stress of about 1.0 tsf corresponding to the point at which the soaked and as-compacted curves cross. This is the equivalent of about 16.5 feet of overburden pressure. Soil above a depth of 16.5 feet may exhibit swelling while soil below a depth of 16.5 feet may exhibit collapse settlement upon wetting. For this embankment, the zone subject to wetting-induced collapse would be from a depth of 16.5 feet to the embankment bottom at 25 feet. Predicted collapse settlement is about 0.4 inches for complete wetting of the soil zone below a depth of 16.5 feet. In Figure 5.26, natural moisture contents in the fill are generally low with an average around 13%. While it is possible that prolonged drought conditions may have caused some drying prior to sampling, it seems little or no substantial increases in moisture content occurred in the fill after construction. Given the small amount of settlement expected for complete wetting of the fill, it seems unlikely that wetting-induced collapse settlements contributed to the observed approach slab settlement. It seems in this case that the other factors mentioned previously are the greatest contributors to the problem.

Table 5.6 Tecumseh Road over Interstate 35 Summary

Tecumseh Road over Interstate 35		**NBI: 24822**	**Constructed:**	**1997**
County:	Cleveland		*ODOT Division:*	3
Abutment Type:	Non-Integral		*Traffic Direction:*	E & W
Site Description:				
Embankment Thickness:	25'-30'	*Embankment Description:*	Unclassified borrow	
Natural Deposit Thickness:	10'	*Natural Soil Description:*	Predominately fat clay	
Bedrock:	Shale			
Geology:	Permian Age Hennessy Shale			
Site Condition:				
Driving Surface:	Mild	*Likely issues:*	Erosion, lateral spreading of the wing-wall, settlement	
Abutment:	Severe movements and cracking			
Under Approach:	Voids			
Maintenance:	Uretek® under slab	*ADT (2008)*	79,200	

Bridge information obtained from ODOT Design Sheets, Bridge Inspection Reports, GRIP Lite, ODOT Geology Manuals, and Geology Map of Oklahoma

Table 5.7 Summary of Oedometer Results for Tecumseh Road over I-35

Boring	Test Type	Depth (ft)	Elev. (ft.)	P'c (tsf)	P'o (tsf)	OCR	Cc	Cr	eo
BH-2	Soaked	16.9	1188.4	1.36	1.36	1.0	0.152	0.023	0.52
BH-3	In Situ	6.4	1198.4	1.57	1.02	1.5	0.223	0.024	0.50
BH-3	Soaked	6.4	1198.4	1.36	1.02	1.3	0.114	0.016	0.48
BH-3	In Situ	26.8	1178.0	1.78	1.71	1.0	0.139	0.016	0.51
BH-3	Soaked	26.8	1178.0	2.61	1.68	1.6	0.172	0.009	0.70
BH-9	In Situ	31.0	1174.4	2.19	1.81	1.2	0.215	0.009	0.64
BH-9*	Soaked	30.5	1174.9	1.57	1.80	1.0	0.214	0.014	0.56

Notes: "In situ" implies samples were tested at natural water content without adding water. "Soaked" implies samples were fully submerged during the test. * OCR equal one since interpreted P'c from oedometer curve is less than P'o

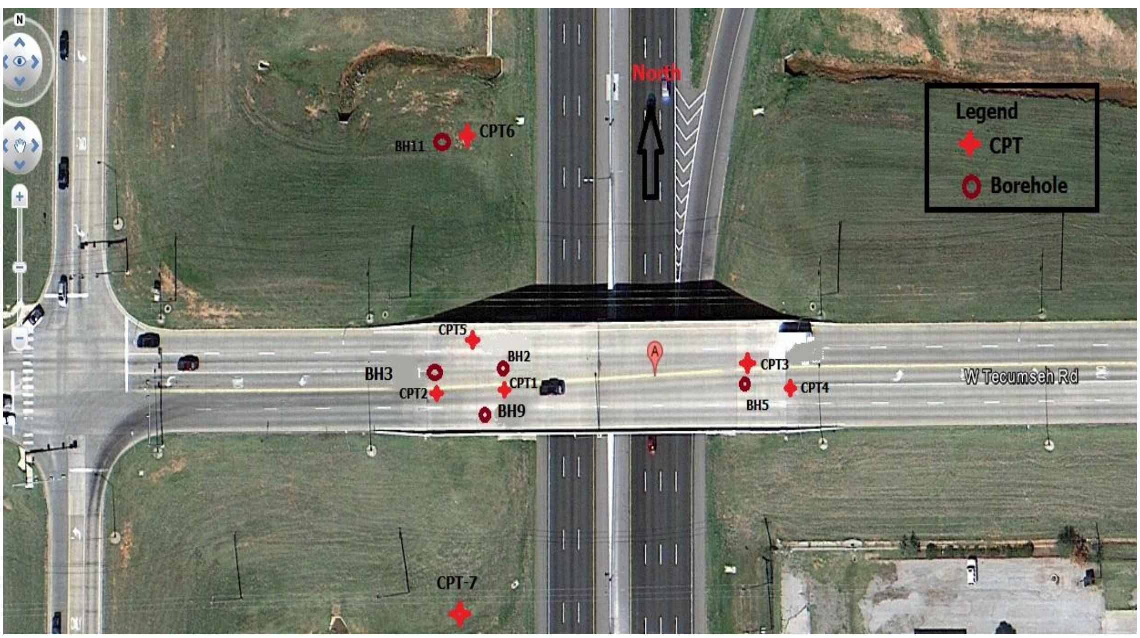

Figure 5.25 Sounding Locations for Tecumseh Road over I-35

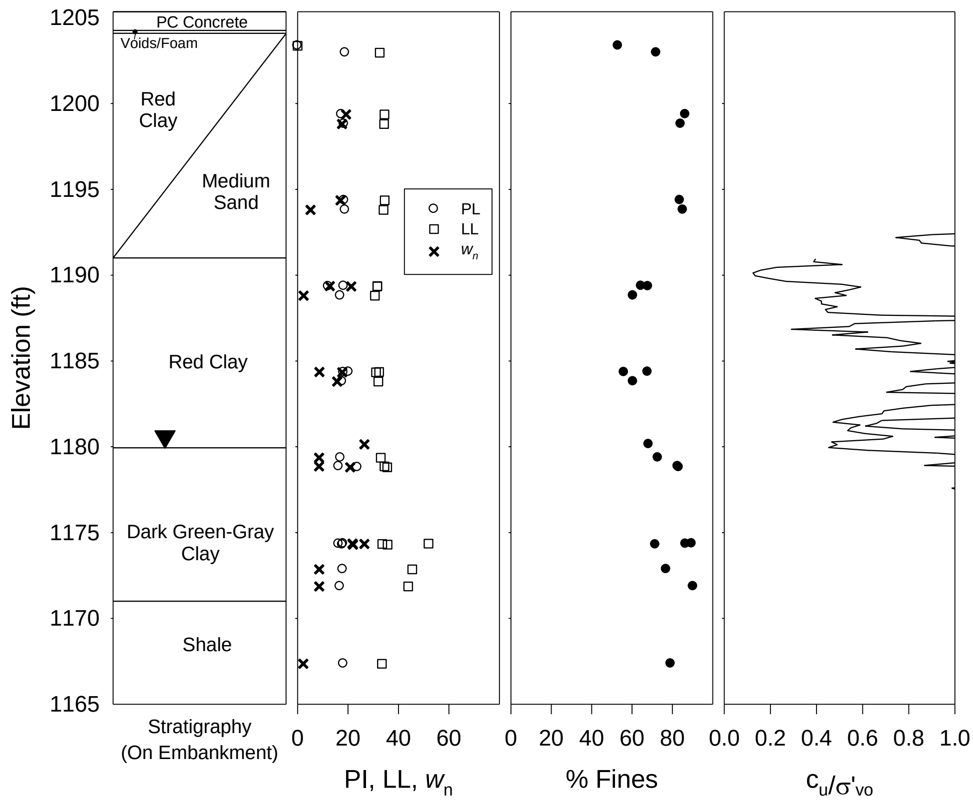

Figure 5.26 Soil Properties for both Embankments –Tecumseh Rd. / I-35

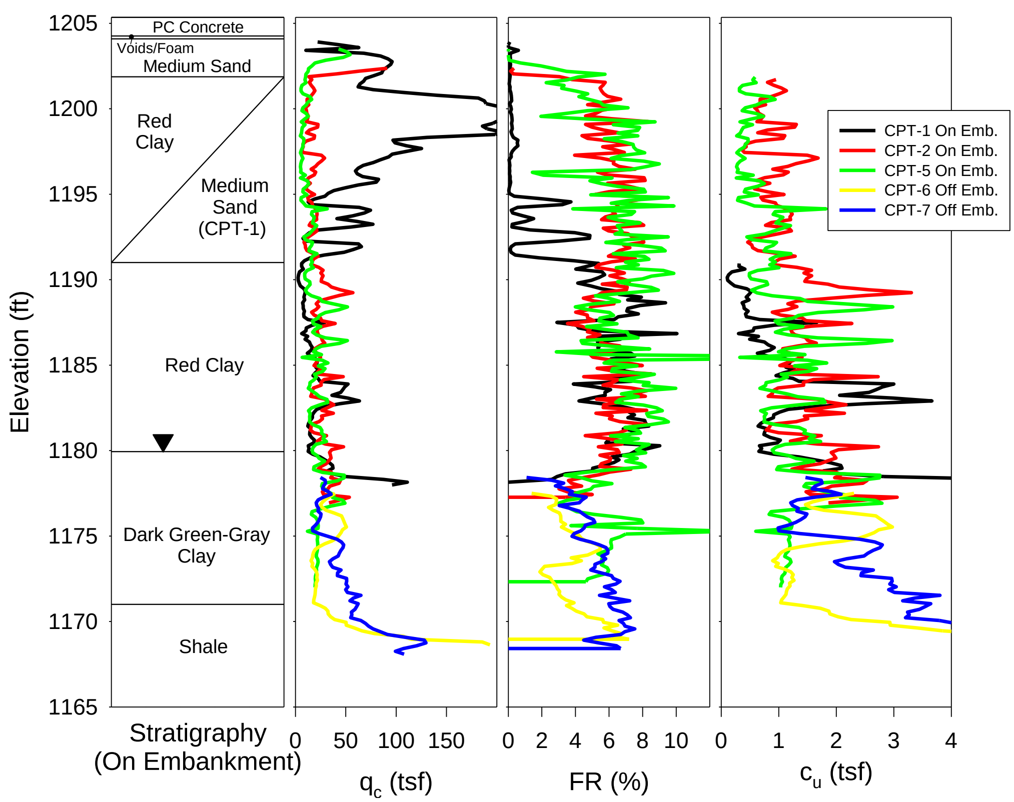

Figure 5.27 Cone Soundings for West Embankment – Tecumseh Rd. / I-35

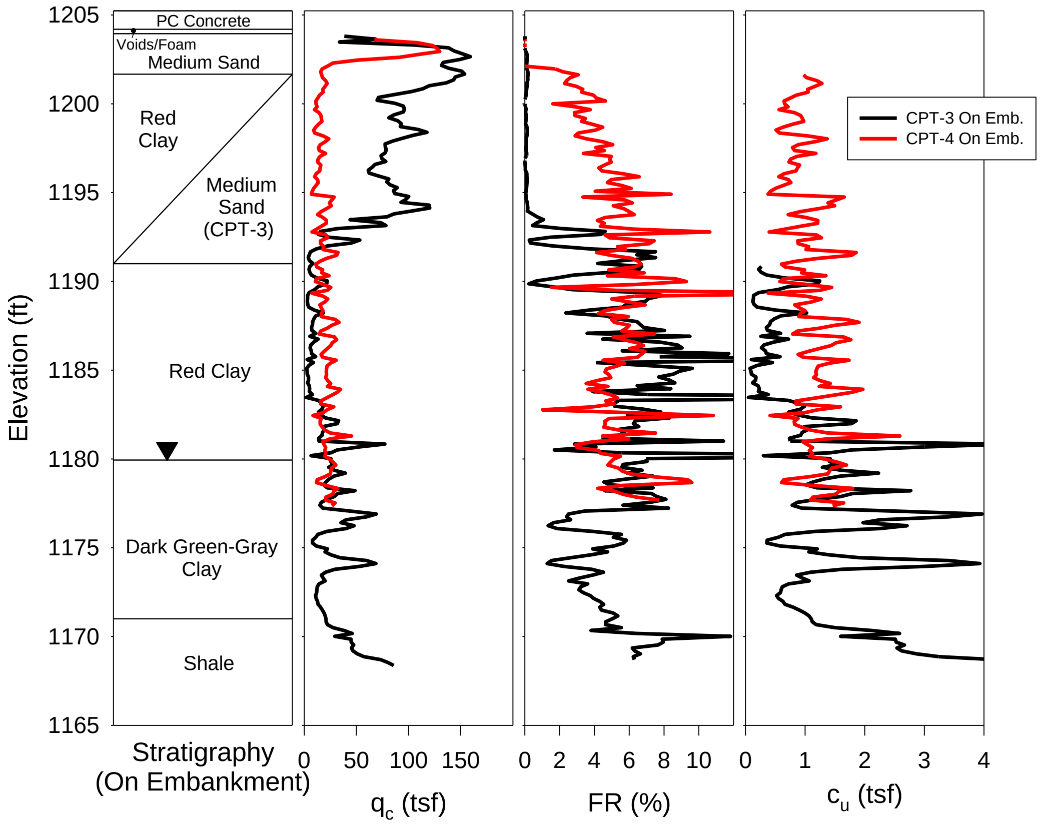

Figure 5.28 Cone Soundings for East Embankment – Tecumseh Rd. / I-35

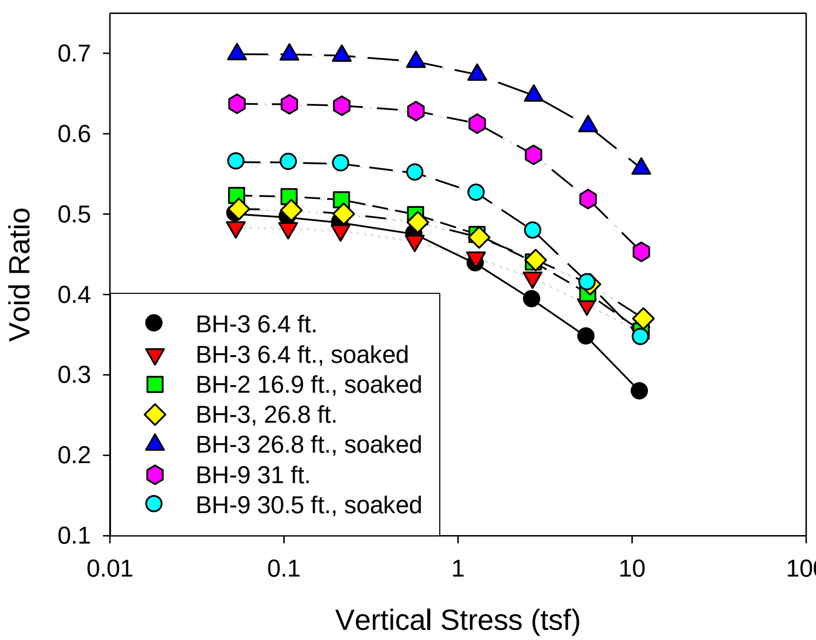

Figure 5.29 Oedometer Test Results – Tecumseh Rd. over I-35

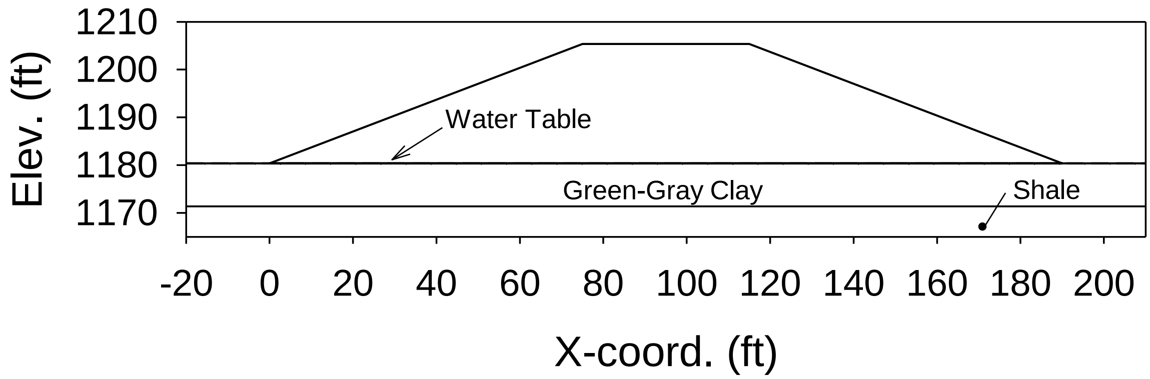

Figure 5.30 Embankment Geometry and Soil Profile Assumed for Settlement Analysis – Tecumseh Rd. over I-35

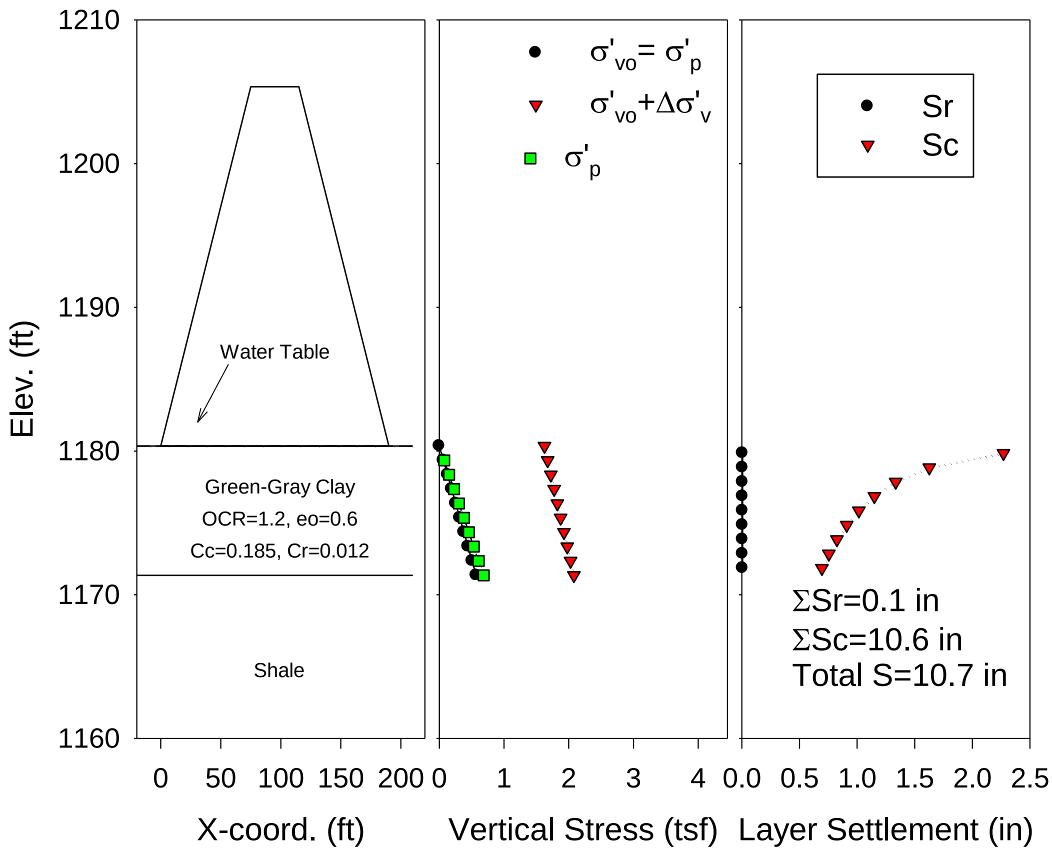

Figure 5.31 Results of Settlement Analysis – Tecumseh Rd. over I-35

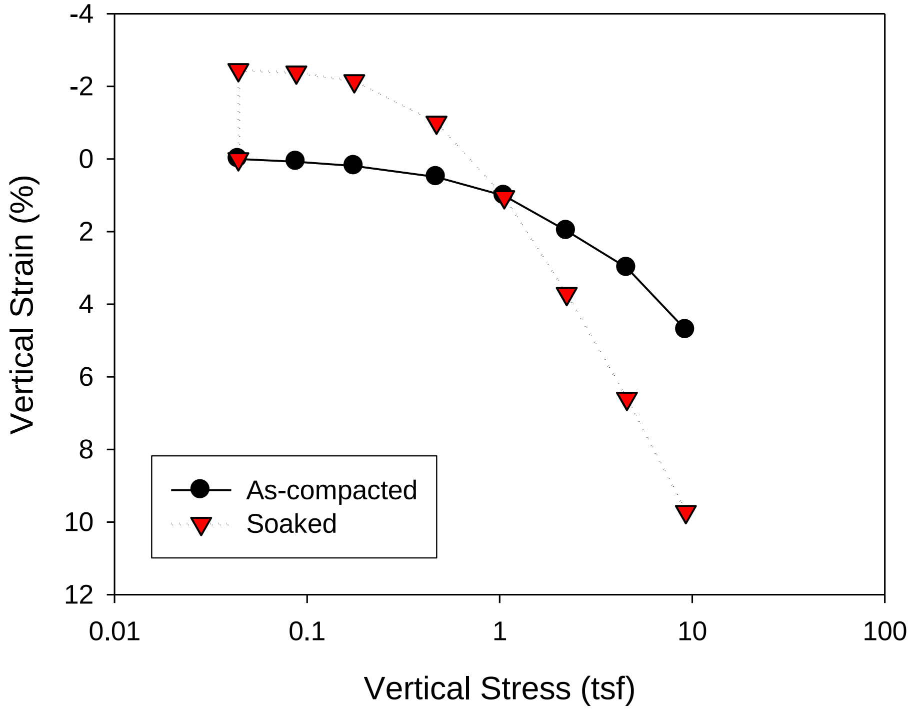

Figure 5.32 Double-Oedometer Results – Tecumseh Rd. / I-35: 6-14 feet

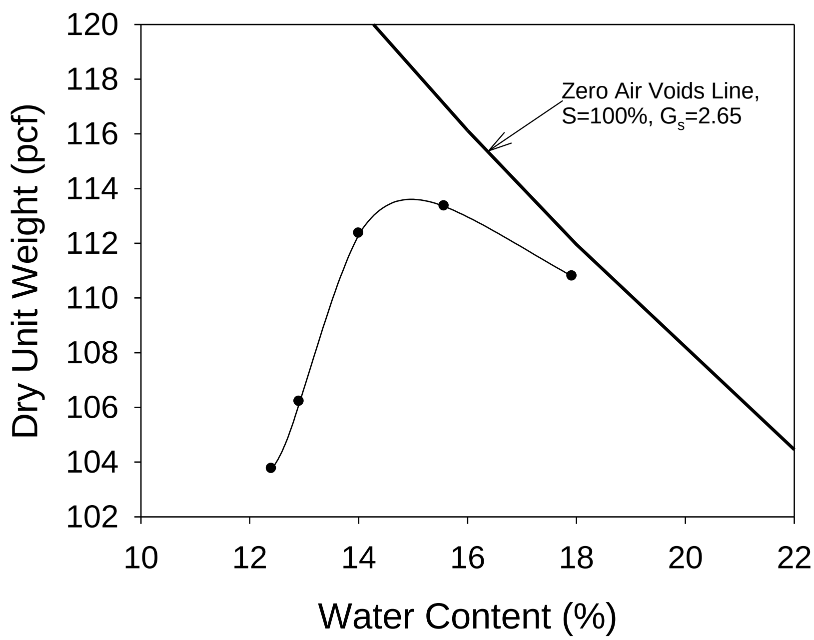

Figure 5.33 Standard Compaction Curve – Tecumseh Rd. / I-35: 6-14 feet

5.6 STATE HIGHWAY 1 OVER STATE HIGHWAY 99

This is an integral abutment bridge constructed in 2001. The maintenance records indicate settlement and leaching through the abutment. Mudjacking has been performed on the east approach to maintain a smooth driving surface. Table 5.8 presents a summary of the bridge information.

5.6.1 Results of Subsurface Investigation

Test boring and CPT sounding locations are shown in Figure 5.34. Soil properties and interpreted stratigraphy are presented in Figures 5.35 and 5.36 for the east and west embankments, respectively. Interpreted stratigraphy is based on results of test borings and CPT profiles shown in Figures 5.37 and 5.38. Based on the project drawings and subsurface profiles it appears the embankment soils consist of medium sand backfill behind the abutments and clayey sand for the remainder of fill. Based on project drawings it appears the embankment soils are about 10 feet thick on the west side and nearly 20 feet thick on the east side. Natural soils are quite variable below the west embankment and made up of clayey sand although weathered layers of sandstone, shale and limestone were encountered. Natural soils below the east embankment appear more uniform and consist of primarily of clayey sand. It appears based on the borings and CPT profiles for the east embankment that the natural soils are at least 13 feet thick. Results of CPT-1 show an increase in tip resistance at about 13 feet, which may indicate the beginning of the weathered rock zone. However, the CPT-4 sounding did not yet encounter rock at this depth as shown in Figure 5.37.

Based on the preceding discussion, for the purpose of analyzing the settlement of the foundation soil a thickness of 15 feet was assumed below the east embankment. Consolidation test results are presented in Table 5.9 and Figure 5.39. The assumed embankment geometry and soil properties used in the analysis are shown in Figures 5.40 and Figures 5.41. Settlement parameters used for the analysis, shown in Figure 5.41, are based on the average values representing natural soils below the embankments corresponding to borings BH-4, BH-5, and BH-6 in Table 5.9. These parameters are consistent with other Oklahoma soils as shown in Figures 5.5 and 5.6, although recompression parameters for the higher PI soils are higher than noted for other sites. It should be noted the higher recompression indices were obtained from unload-reload curves for these particular higher PI soils.

Results of the settlement analysis predict a total settlement of 9.4 inches, being mostly due to virgin compression. Estimates of wetting-induced compression settlement based indicate a possible collapse settlement of about 0.4 inches for complete wetting of a 20-foot thick embankment. Wetting-induced compression was calculated based on the double-oedometer results shown in Figure 5.42 for a sample compacted to about 97% relative compaction and 2 percentage points dry of the OMC based on the standard compaction curve shown in Figure 5.43. Since wetting-induced collapse is only expected to occur in the lowest 5 feet of the embankment and collapse strains in the corresponding stress range are relatively small, predicted collapse settlements are relatively low for this fill. Results of the settlement analysis suggest the primary settlement mechanism for this site is consolidation of the foundation soils. In addition, voids were detected under the approach slabs suggesting that possibly some erosion of the soils beneath the slabs may be occurring.

Table 5.8 State Highway 1 over State Highway 99 Summary

State Highway 1 over State Highway 99		**NBI: 26915**	**Constructed:**	**2001**
County:	Pontotoc		*ODOT Division:*	3
Abutment Type:	Integral		*Traffic Direction:*	E & W
Site Description:				
Embankment Thickness:	West: 10’ East: 20’	*Embankment Description:*		
Natural Deposit Thickness:	10-20’	*Natural Soil Description:*	Predominately stiff lean clay	
Bedrock:	Sandstone over Shale			
Geology:	Pennsylvanian Age Ada Unit			
Site Condition:				
Driving Surface:	Mild	*Likely issues:*	Embankment settlement Erosion	
Abutment:	Mild leaching			
Under Approach:	Void, filled			
Maintenance:	Mudjacking	*ADT (2008)*	11,800	

Bridge information obtained from ODOT Design Sheets, Bridge Inspection Reports, GRIP Lite, ODOT Geology Manuals, and Geology Map of Oklahoma

Table 5.9 Summary of Oedometer Results for SH-1 over SH-99

Boring	Test Type	Depth (ft)	Elev. (ft)	P'c (tsf)	P'o (tsf)	OCR	Cc	Cr	eo
BH-2	Soaked	11.0	990.6	2.04	1.31	1.6	0.222	0.015	0.63
BH-3*	In Situ	11.4	990.1	1.15	1.32	1.0	0.121	0.019	0.49
BH-3	Soaked	11.8	989.7	2.19	1.33	1.6	0.174	0.012	0.40
BH-4	In Situ	14.0	994.6	1.57	1.41	1.1	0.118	0.012	0.49
BH-4	Soaked	14.8	993.8	2.14	1.44	1.5	0.224	0.013	0.64
BH-5^	In Situ	5.5	980.7	3.24	1.12	2.9	0.179	0.038	0.49
BH-5^	Soaked	5.4	980.8	1.78	1.12	1.6	0.169	0.057	0.59
BH-6^	Soaked	6.0	966.7	1.78	1.14	1.6	0.138	0.048	0.49

Notes: “In situ” implies samples were tested at natural water content without adding water. “Soaked” implies samples were fully submerged during the test. * OCR equal one since interpreted P'c from oedometer curve is less than P'o, ^ Cr from unload-reload loop

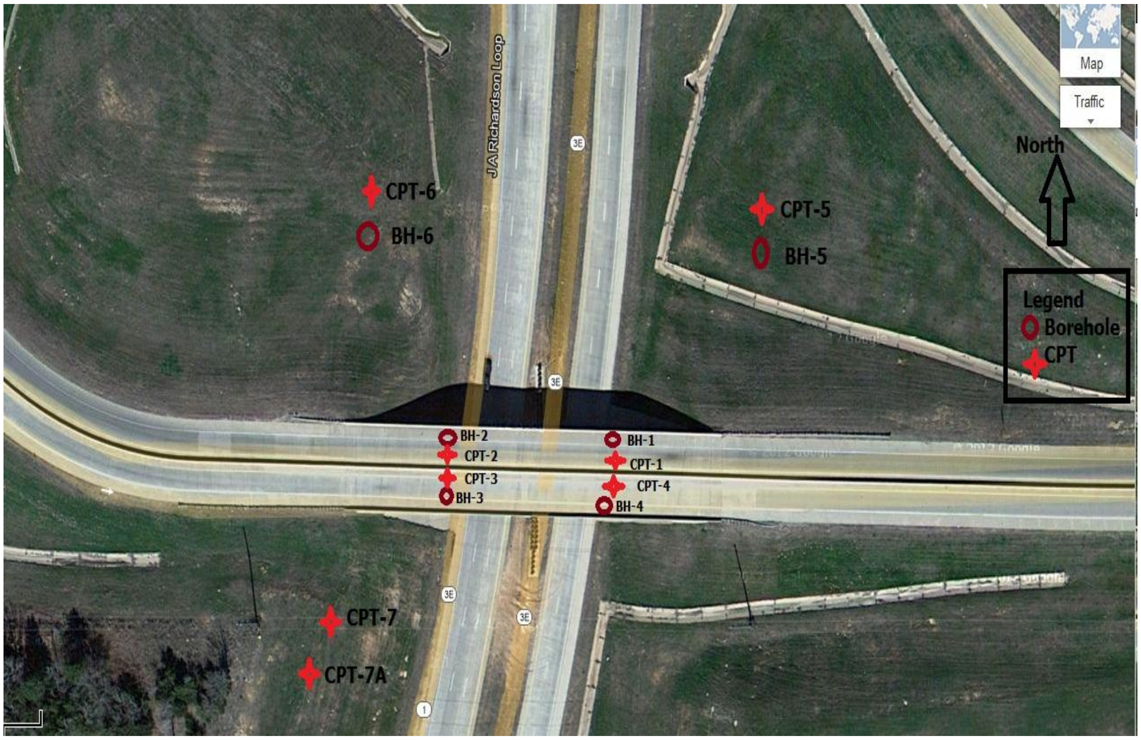

Figure 5.34 Sounding Locations for SH-1 over SH-99

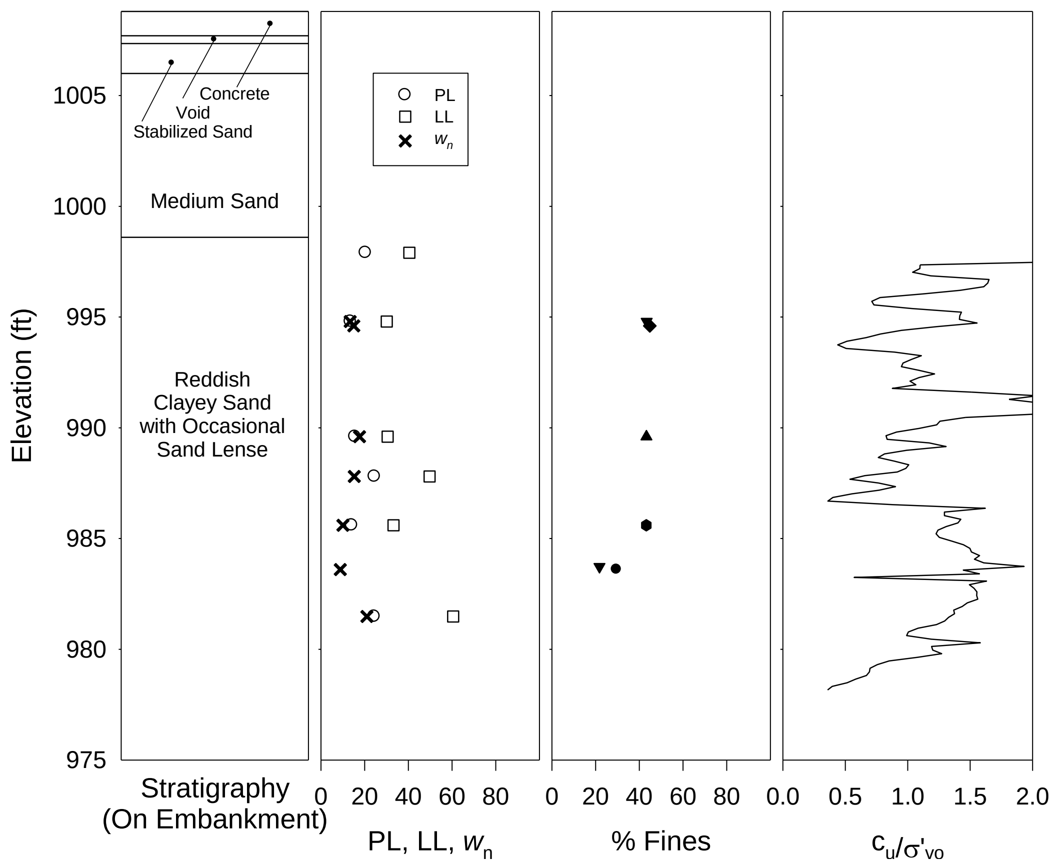

Figure 5.35 Soil Properties for East Embankment – SH-1 over SH-99

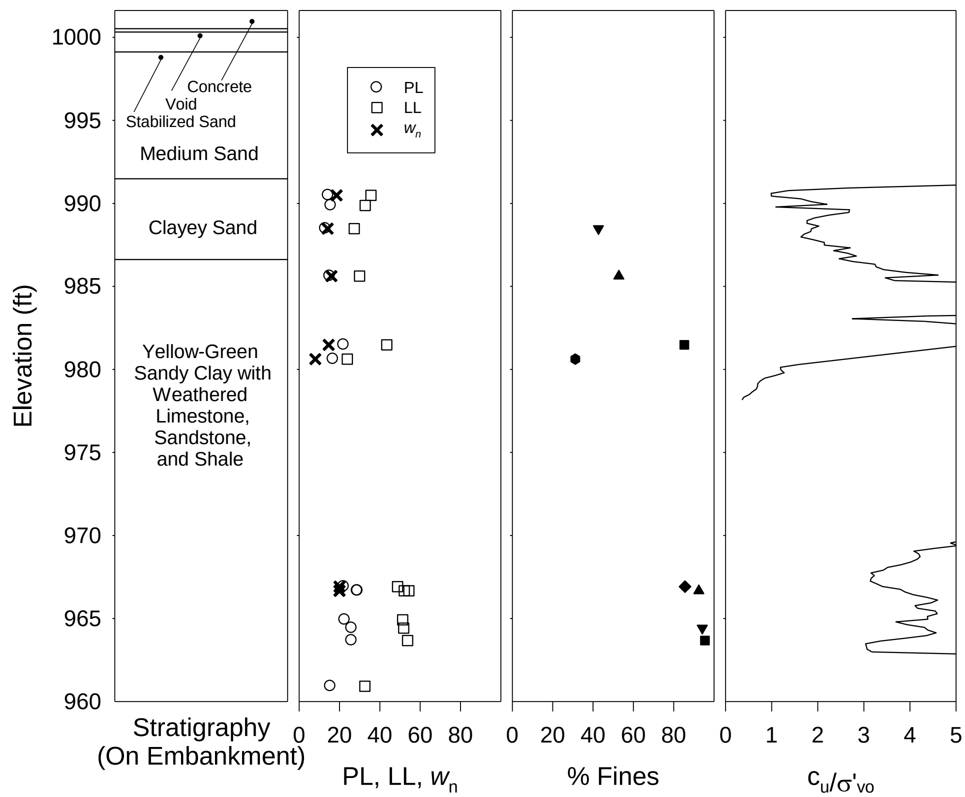

Figure 5.36 Soil Properties for West Embankment – SH-1 over SH-99

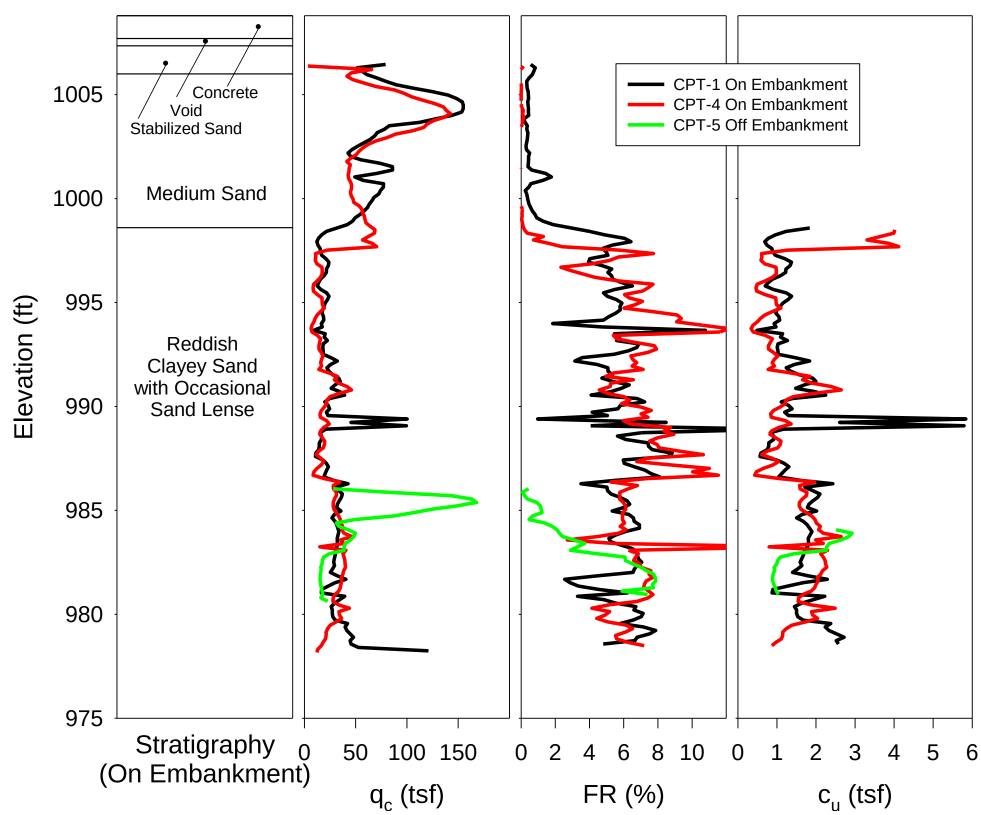

Figure 5.37 Cone Soundings for East Embankment – SH-1 over SH-99

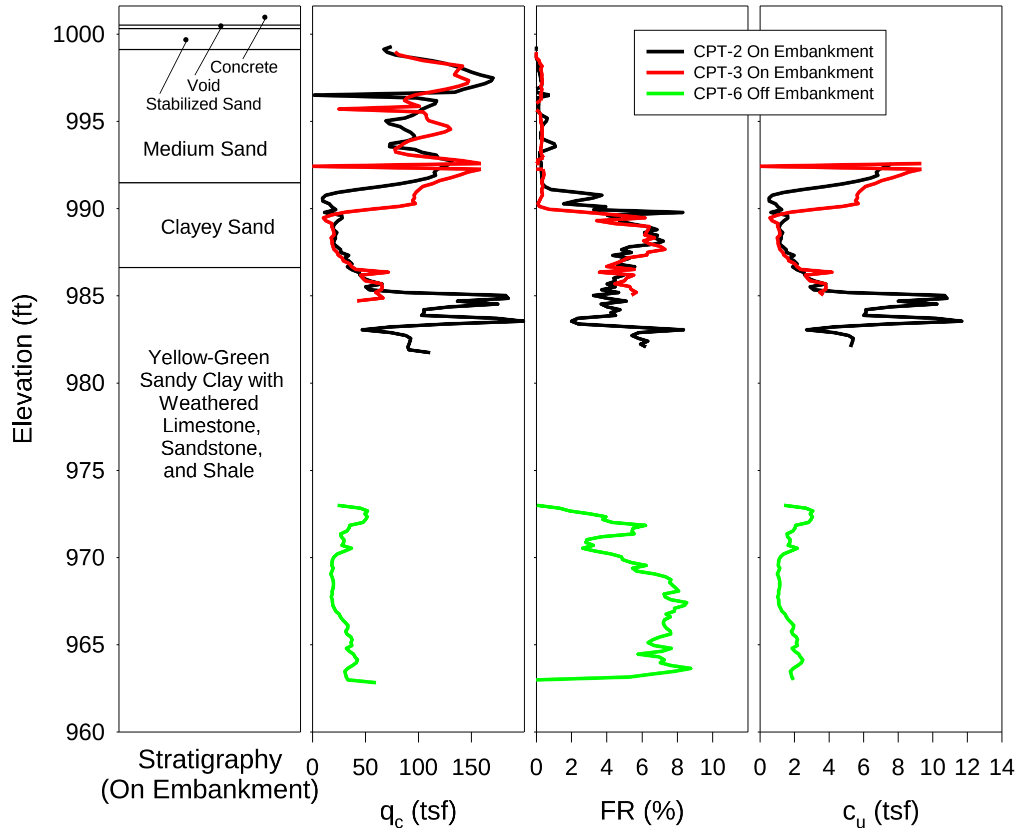

Figure 5.38 Cone Soundings for West Embankment – SH-1 over SH-99

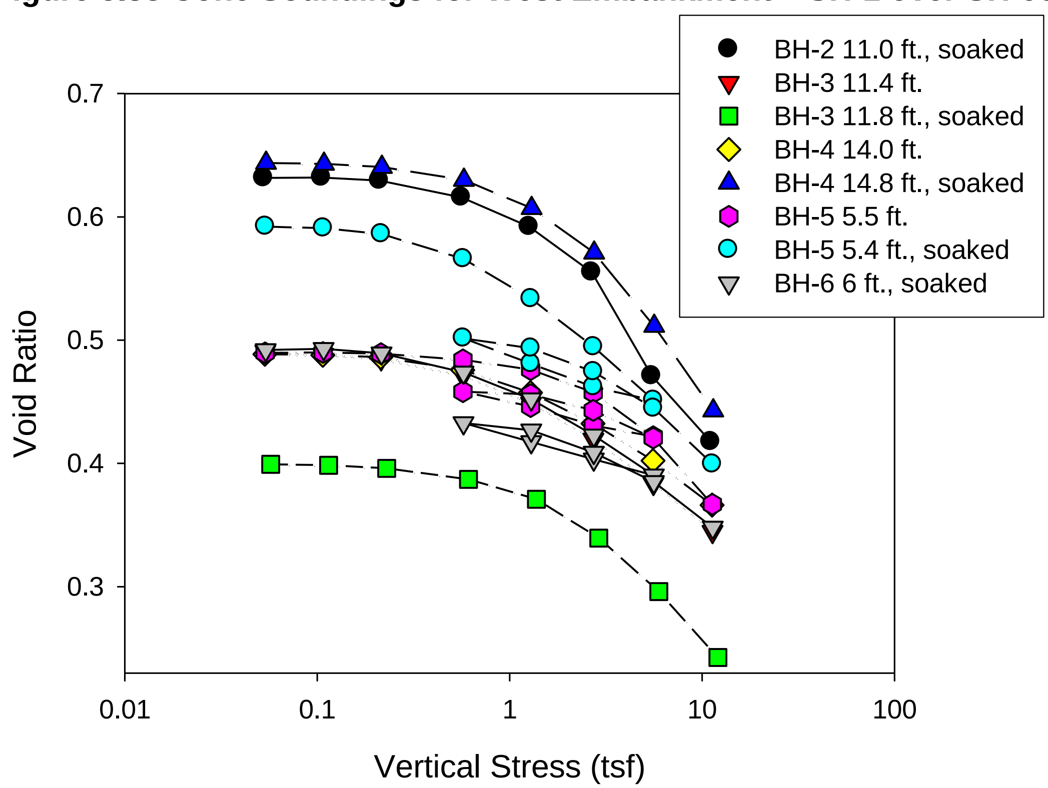

Figure 5.39 Oedometer Test Results – SH-1 over SH-99

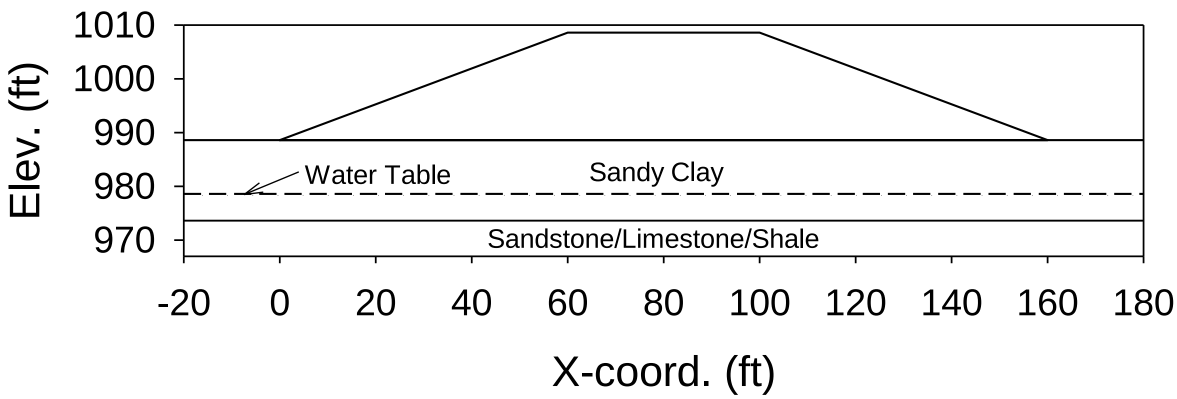

Figure 5.40 Embankment Geometry and Soil Profile Assumed for Settlement Analysis – SH-1 over SH-99

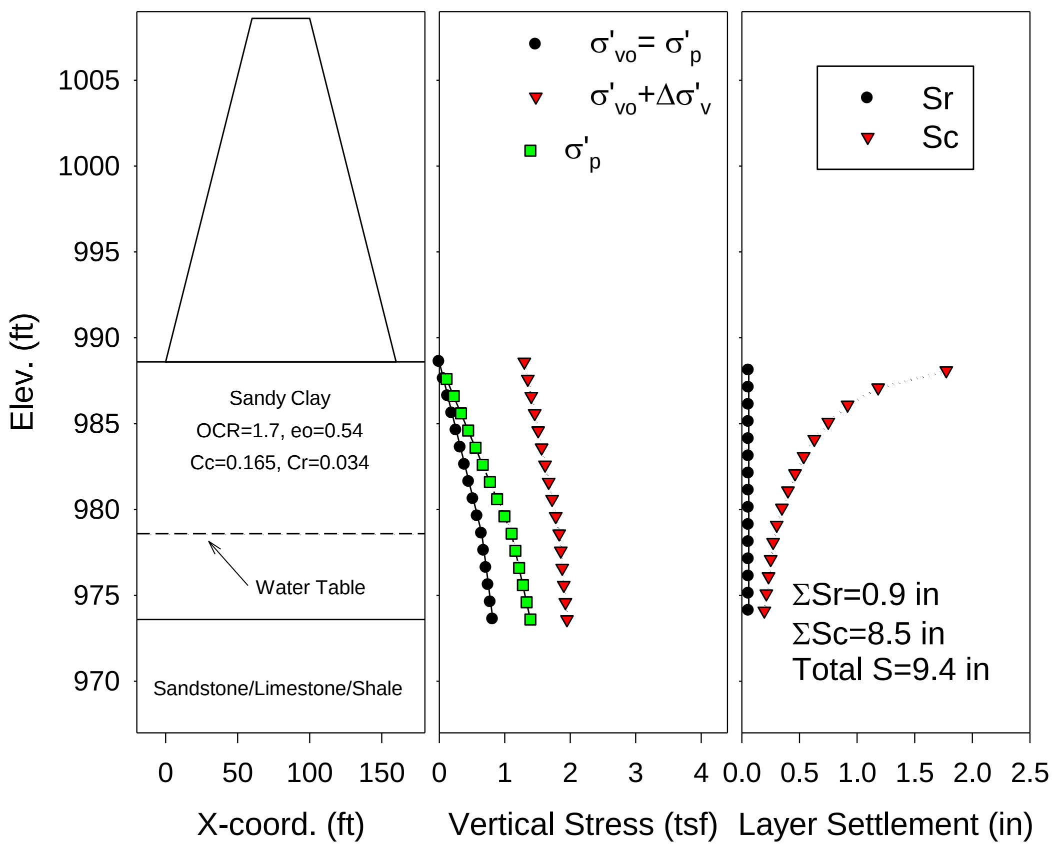

Figure 5.41 Results of Settlement Analysis – SH-1 over SH-99

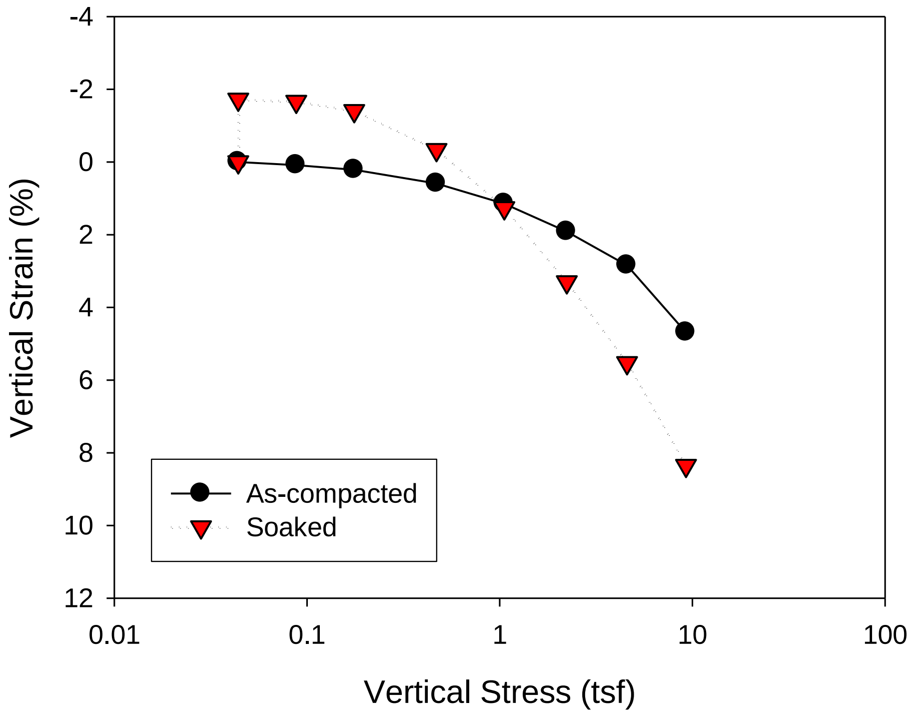

Figure 5.42 Double-Oedometer Results – SH-1 over SH-99: BH-4 22-26 feet

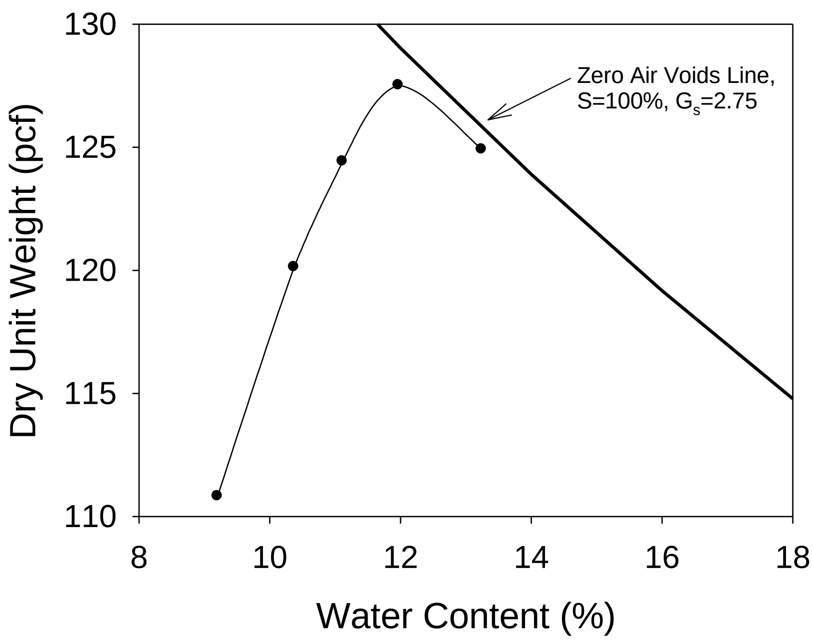

Figure 5.43 Standard Compaction Curve – SH-1 over SH-99: BH-4 22-26 feet

Chapter 6 SUMMARY AND RECOMMENDATIONS

6.1 SUMMARY OF FINDINGS

Results of this study have revealed that bridge approach settlement is a multifaceted problem. A surprising number of the sites investigated revealed significant problems with drainage and erosion. Of particular concern is loss of soil from beneath approach slabs and in many cases evidence of significant erosion under abutments. Both of these mechanisms can lead to formation of voids and approach slab settlement. What is needed is a complete rethinking of design of surface and sub-drainage systems to efficiently remove water from beneath pavements and backfill while diminishing or eliminating the erosive power of the water. Pavement surface joint seals were largely compromised and so the subsurface drainage must be designed with the expectation that water will enter the joints, potentially with enough erosive power to undermine approach slabs. Some ideas for addressing drainage and erosion issues are presented in the recommendations section below.

In addition to erosion, there is significant potential for foundation settlement below some embankments. Four embankments subjected to subsurface exploration and laboratory testing were analyzed for foundation settlements. The analysis resulted in estimated settlements, based on average properties, of 5 to 11 inches. For three of these sites, predictions of settlement exceeded 9 inches. Settlements of this magnitude are certainly problematic for approach slabs. Even if a portion of this settlement occurs during construction, settlements of even a few inches can be problematic if not accounted for in design and other factors such as erosion compound the problem. Of particular importance is the observation that even modestly high embankments, 7 feet in the case of SH-59 over Big Creek, can result in significant settlement when placed over thick modestly compressible soil profiles. A particular concern is the presence of compressible cohesive layers close to the bottom of the embankment where stress increases are significant. Even relatively thin layers, on the order of a few feet can pose a problem. For the embankments investigated, settlement due to post construction wetting did not appear to be a major factor; however, this potential should not be overlooked where fill heights are significant and post construction wetting is likely. Prior research has shown that this mechanism can be problematic (Miller et al. 2001, Lim and Miller 2004).

In one case investigated, Tecumseh Road over I-35, the settlement problem seemed partly related to an outward movement of the wing walls. Significant lateral movement is accompanied by downward movement of the adjacent soil. In addition, such movements open up joints between adjacent structures allowing water to enter the underlying soil and backfill contributing to the erosion problems observed.

6.2 RECOMMENDATIONS

Many highway embankments suffer from bridge approach slab settlement. The results of this research suggest the problem in Oklahoma is due to two main factors. First, is undermining of the approach slab resulting from erosion. In this case water enters poorly sealed pavement joints and other separations. In some cases it may flow laterally and carry soil out the sides of the pavement, while in others it may enter the abutment backfill and flow beneath the abutment where a hydraulic short circuit has developed. The second major problem observed to cause approach slab settlement in this study is compression of the foundation soils. In previous studies, compression of the fill material was also noted to be problematic, which can be caused by construction deficiencies or post-construction wetting. Following are some design and construction recommendations to address the bridge approach slab settlement problem.

1) Standard details for each style of bridge abutment should be carefully examined to identify possible deficiencies such as with the drainage system as noted in Section 4.3 of this report. Critically reviewing standard details should become a standard practice especially where problems with a particular bridge type are recurring.

2) For the case of abutments, backfill drainage systems that potentially could result in a hydraulic short circuit, such as the standard detail for integral abutment bridges (Figure 4.10), should be redesigned to eliminate this problem. Examples of a modified drainage system for the integral abutment detail are shown in Figures 4.38 to 4.41.

3) In one case investigated, Tecumseh Road over I-35, outward lateral movement of the wing walls was noted. In cases where lateral earth pressures may cause wing wall movements the walls should be carefully analyzed to be sure lateral resistance to movement is adequate. Where passive earth pressures are used to provide some portion of the lateral resistance,

consideration must be given to the fact that large lateral movements of the wall are necessary to mobilize full passive resistance. Thus, to limit lateral deformations only a fraction of the full passive resistance is available.

3) Where the approach slab joins the bridge and at other locations where undermining of the slab can occur due to water flowing laterally beneath the slab and in joints or along the edge of the pavement, details should be modified to minimize the soil loss. An example of this type of problem is shown in Figure 4.7. A possible solution would be to provide an erosion resistant filtering material such as sand and gravel in this area or other form of edge draining system. The system should have longevity and properly filter and contain soils underlying the approach slab.

4) The position of drain outlets should be carefully considered for each project. Generally, these outlets should not be placed on slope walls where they potentially could cause erosion and undermining of the slab (such as shown in Figure 4.34). One solution may be to outlet the drains near the bottom of the slope wall into an erosion resistant drainage way.

5) In many cases, drain outlets were found to be blocked and non-functioning. Drain outlets should be designed to be robust, long lasting and free of vegetation. In addition, they should be checked and maintained on a regular schedule.

6) Surface water flowing off the pavement, in the median, and from any other sources, that has the potential to flow over and adjacent to the bridge structures and embankment should be carefully addressed during the design stage.

Erosion along the base of wing walls and edges of approach slabs is a common observation. In extreme cases, uncontrolled surface water drainage has caused severe erosion around the edges of abutments and under slope walls for example. Even concrete lined surface water drainage channels, such shown in Figure 4.4, can be undermined by flowing water. In areas known to be susceptible to erosion, such as the edges of slope walls and at the base of wing walls, erosion resistant materials should be used in addition to providing adequate surface drainage to divert water away from erosion susceptible areas. One possibility would be to install geotextile wrapped gravel drains at these locations to safely convey water to the bottom of the slope. Where erosion is observed to occur at existing bridges, remedial action should be employed to fix surface water drainage systems.

7) Every bridge project involving construction of embankments should involve a complete settlement analysis. Settlement analyses should include evaluation of foundation settlement and embankment soil (fill) settlement. The latter should address both self-weight compression and wetting-induced compression. In addition, if high PI soils are present the potential for swelling should be evaluated (more of a problem for the pavement than the approach slab that typically sits on engineered fills like granular backfill or CLSM). Shelby tube sampling of cohesive soils, or piston tube sampling of soils that are soft, should be accompanied by oedometer testing to obtain consolidation settlement parameters. If for some reason oedometer testing is not conducted, then compression indices can be estimated using empirical methods (e.g. Figures

5.5 and 5.6). This will also require estimating the overconsolidation ratio of the layers in question, which can also be done empirically.

8) Where foundation settlement is expected to be excessive, many techniques are available, some described in the literature review, to reduce the settlement. These might include reinforcement of the foundations soils using rammed aggregate piers or soil-cement columns (Cement Deep Soil Mixing), geosynthetics reinforcement, or surcharging (with or without wick drains).

9) Approach slab design should encompass the likelihood that at least some amount of settlement will occur. As mentioned in the literature review, approach slabs could be strengthened structurally to provide greater bending resistance and to better handle the likelihood of voids under the slab. In addition, it seems a longer approach slab would result in less angular rotation in the likely event of embankment settlement. The use of two shorter slabs does not seem to accommodate the settlement very well as it was observed in some cases that settlement occurred at the joint causing a v-shape profile.

10) It is well known that during construction, quality control is critical during compaction of fill and placement of drainage systems. Frequent in place testing of density and moisture content of compacted materials should be performed and compaction specs should be rigidly enforced. In addition, evaluation of the fill material should be done routinely to be sure it is consistent with expectation and the compaction reference standards (OMC and maximum dry density) being used. The presence of oversize material should be noted, quantified, and incorporated into the compaction quality control process. If the amount of

oversize material changes in the field, then a new corrected reference density and moisture content must be used.

While the aforementioned process and recommendations are well known and appreciated by ODOT engineers, it seems occasionally this process is not followed during construction and results in substantial settlements due to poorly compacted fill materials. It seems prudent to routinely review and evaluate the quality control process throughout the organization and its various divisions and residencies to be sure earthwork procedures are being properly followed.

11) In the case of high embankments (say greater than 15 feet), the potential for fill compression resulting from poor compaction or post construction wetting increases with embankment height. For example, in properly compacted soils compacted dry of optimum (say 97% relative compaction, OMC-2%), wetting-induced compression may become substantial for some soils roughly 15 feet below the top of the embankment and deeper. This depth essentially corresponds to the overburden pressure where most soils begin to exhibit collapse behavior. If substantial post-construction wetting of high embankments is expected in the lower regions of the embankment, then designers should perform a collapse settlement analysis. Examples may include areas where groundwater may impact the embankment, flooding may occur (culverts during large rainfall events), water crossings, or seasonal rainfall is significant. In cases where collapse settlements or self-weight compression of the fill are expected to be excessive, designers may employ various techniques to reduce the settlement potential such as mechanical stabilization (e.g., use relative

compaction of 95% of modified or 98% standard) or chemical stabilization (e.g. lime, fly ash, cement kiln dust). These techniques can be selectively applied to the fill at greater depths in the embankment where collapse potential may be significant. Another possibility would be to use properly compacted granular soils that are less susceptible to collapse.

12) The use of a relative density based specification for granular backfill will ensure more adequate compaction, as opposed to a standard Proctor based specification. Previous work (e.g. Siemers 2004 and Miller and Snethen 2006) has shown that granular soils compacted to standard Proctor density often achieve relative densities much lower than the suggested target of 65% for granular soils.

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Appendix A: List of Bridges Sites Considered for Analysis

Item	Intersection	County	ODOT Division	Over
1	19th Street over I-35	Cleveland	3	Road
2	E-1350 Road over Lake Eufala	Pittsburgh	2	Water
3	Hereford Lane over SH-69	Pittsburgh	2	Road
4	I-35 over Main	Cleveland	3	Road
5	I-44 over Medicine Bluff Creek	Comanche	7	Water
6	SH-1 over BNSF Railroad	Pontotoc	3	Road
7	SH-1 over SH-99	Pontotoc	3	Road
8	SH-11 over I-35	Kay	4	Road
9	SH-152 over Lake Creek	Caddo	7	Water
10	SH-152 over Willow Creek	Caddo	7	Water
11	SH-3 over Chiquita Creek	Texas	6	Water
12	SH-3 over Clear Creek	Beaver	6	Water
13	SH-3 over Duck Pond Creek	Beaver	6	Water
14	SH-3 over Fulton Creek	Beaver	6	Water
15	SH-3 over Kiowa Creek	Beaver	6	Water
16	SH-3 over Palo Duro Creek	Texas	6	Water
17	SH-3 over S. Fork Clear Creek	Beaver	6	Water
18	SH-3 over Unnamed Creek	Beaver	6	Water
19	SH-31 over Muddy Boggy	Coal	3	Water
20	SH-31 over Salt Creek	Coal	3	Water
21	SH-33 over Fitzgerald Creek	Logan	4	Water
22	SH-39 over Willow Creek	Cleveland	3	Water
23	SH-3W over Big Creek	Pontotoc	3	Water
24	SH-48A over Blue River	Johnston	3	Water
25	SH-59A over Big Creek	Pontotoc	3	Water
26	SH-59B over Coon Creek	Pottawatomie	3	Water
27	SH-6 North of Retrop	Washita	5	Water
28	SH-6 over Sadler Creek	Beckham	5	Water
29	SH-6 over West Elk Creek	Beckham	5	Water
30	SH-7 over Beaver Creek	Comanche	7	Water
31	SH-9 over Running Creek	Caddo	7	Water
32	SH-99 over Blue River	Johnston	3	Water
33	Shields Blvd. over I-35	Cleveland	3	Road
34	Tecumseh Road over I-35	Cleveland	3	Road

35	US-177 over Salt Fork River	Noble	4	Water
36	US-183 over Boggy Creek	Washita	5	Water
37	US-183 over Sand Creek	Woodward	6	Water
38	US-259 over Kiamichi River	LeFlore	2	Water
39	US-412 over Indian Creek	Major	6	Water
40	US-412 over Main Creek	Major	6	Water
41	US-62 over Robinson Creek	Lincoln	3	Water
42	US-64 over White Horse Creek	Woods	6	Water
43	US-75 over Muddy Boggy	Coal	3	Water

Appendix B: Test Boring Logs

SH-59A over Big Creek

Borehole Log	
Project:	ODOT Approach Slab Settlement
Location:	SH-59A over Big Creek
Date:	6/21/2012
Sounding:	B-1
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine
Ground Surface Elev. (ft)	982.84

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.15						982.69	Asphalt Concrete	
0.15	1.2						981.64	Portland Cement Concrete	
1.2	1.5						981.34	Void	
1.5	2							Coarse to Medium Sand	
2	3.5	3	13	100				Coarse to Medium Sand	
3.5	5						977.84	Coarse to Medium Sand	
5	6.5	6	3	100				Reddish Brown Clay	
6.5	10							Reddish Brown Clay	
10	12				Yes	40		Reddish Brown Clay	No cuttings return because of voids
12	13							Reddish Brown Clay	
13	15				Yes	65		Reddish Brown Clay	
15	20						962.84	Reddish Brown Clay	
20	22				Yes	100		Very dark Grey Clay	No cuttings return, color change noted on drag bit
22	25							Very dark Grey Clay	
25	27				Yes	100		Dark Reddish Brown Clay	

Borehole Log

Project:	ODOT Approach Slab Settlement
Location:	SH-59A over Big Creek
Date:	6/20/2012
Sounding:	B-2
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine

Ground Surface Elev. (ft) 982.78

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.1						982.68	Asphalt Concrete	
0.1	1.1						981.68	Portland Cement Concrete	
1.1	1.3						981.48	Void	
1.3	2							Sand	
2	3.5	3	28	100				Coarse to Medium Sand	
3.5	5							Coarse to Medium Sand	At 4' became more moist
5	6.2							Coarse to Medium Sand	
6.2	6.5	6	3	100				Coarse to Medium Sand	
6.5	7.5						975.28	Coarse to Medium Sand	
7.5	10							Reddish Brown Clay	
10	12				Yes	75		Reddish Brown Clay	
12	15							Reddish Brown Clay	
15	17				Yes	90	965.78	Reddish Brown Clay	
17	20							Dark Grey Sandy Clay	
20	21.5	21	10	100				Very Dark Grey Clay	
21.5	25						957.78	Very Dark Grey Clay	
25	27				Yes	100		Dark Grey Clay	wet
27	30							Dark Grey Clay	
30	32				Yes	100		Reddish Brown Clay	wet
32	35							Reddish Brown Clay	
35	37				Yes	95			

Borehole Log

Project:	ODOT Approach Slab Settlement
Location:	SH-59A over Big Creek
Date:	6/19/2012
Sounding:	B-3
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine

Ground Surface Elev. (ft) 968.41

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.5							Silty Clay	
0.5	1							Medium Sand	
1	1.5	1	7					Fine Sand	
1.5	5							Light Brown Sandy Clay	Moist
5	6.5	6	3	100				Dark Brown Clay	Moist
6.5	10							Dark Brown Clay	Moist
10	12				Yes	100		Light to Dark Brown Clay	
12	13							Light Brown Clay	
13	15						953.41	Sandy Clay	Dry
15	17				Yes	80		Light Brown Clay	
17	20							Light Brown Clay	
20	22				Yes	75		Light Brown Clay	
22	25						943.41	Light Brown Clay	
25	27				Yes	88		Brown Clay	
27	35							Brown Clay	

US-177 over Salt Fork

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		US-59A over Big Creek							
Date:		4/4/2012							
Sounding:		B-1							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			932.14						
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.05						931.09	Asphalt Concrete	
1.05	2.5						929.64	Aggregate	
2.5	3							Fine Sand	
3	3.2							Fine Sand	
3.2	3.8							Coarse Sand	
3.8	4.2	4	13					Fine Sand	
4.2	4.5							Fine Sand	
4.5	5							Fine Sand	
5	6							Fine Sand	
6	6.5	6	7	100				Fine Sand	
6.5	7							Fine Sand	more moist
7	10							Fine Sand	
10	10.2						921.94	Fine Sand	
10.2	11.5	11	7					Reddish Clay	
11.5	13							Reddish Clay	
13	15				Yes	95	917.14	Organic Black Clay	
15	16								
16	17							Dark Brown Clay	
17	18							Reddish Clay	
18	20				Yes	100			
20	24							Sandy Clay	with thin layer of sand
24	26				Yes	100			

Borehole Log

Project:	ODOT Approach Slab Settlement
Location:	US 177 over Salt Fork River
Date:	4/4/2012
Sounding:	B-2
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine
Ground Surface Elev. (ft)	929.16

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.9						928.26	Asphalt Concrete	
0.9	2						927.16	Aggregate	
2	3							Silty Fine Sand	
3	4.5	4	22	73					
4.5	5							Silty Fine Sand	
5	6.5	6	24	66					
6.5	8								
8	10				Yes	100			
10	14								
14	15.7				Yes	100			
15.7	18							Silty Sand	
18	19.5	19	36	80				Fine Sand	
19.5	23						906.16		
23	24.5	24	6	60				Clayey Sand	
24.5	28								
28	29.5	29	24	80				Fine to Medium Sand	

Borehole Log

Project:	ODOT Approach Slab Settlement	
Location:	US 177 over Salt Fork River	
Date:	4/10/2012	
Sounding:	B-3	
Rig Type:		
Drilling Method:		Air Rotary
Operator:		Larry Taylor (ODOT)
OU Field Rep.:		Karim Saadedine

Ground Surface Elev. (ft) 932.12

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.05						931.07	Asphalt Concrete	
1.05	2						930.12	Aggregate	
2	3.5	3	9	73				Coarse Sand and Gravel	
3.5	5							S.A.	
5	5.4							Silty Fine Sand	
5.4	5.7							Coarse Sand	
5.7	6.5	6	3	73				Silty Fine Sand	Wet
6.5	10							Silty Fine Sand and Coarse Sand	
10	10.7							Silt Fine Sand	
10.7	11.5	11	10	73				Silty Clay	
11.5	15							S.A.	
15	17				Yes	100		Red to Brown Silty Clay	
17	20							Red Silty Clay	
20	22				Yes	100		Red Clay	
22	22.5							Red Clay	
22.5	25							Brown Silty Sand with Organic Material	
25	25.3							Brown Silty Sand	
25.3	26.5	26	15	100				Brown Clay	
26.5	29							Brown Clay	
29	30							Red Clay	
30	31.5	31	13	100				Red Clay	

Borehole Log									

Project:	ODOT Approach Slab Settlement
Location:	US 177 over Salt Fork River
Date:	4/10/2012
Sounding:	B-4
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine

Ground Surface Elev. (ft) 929.28

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.1						928.18	Asphalt Concrete	
1.1	2						927.28	Aggregate	
2	2.2							Silty Fine Sand	
2.2	3	3	15	87				Coarse Sand	
3	3.3							Silty Fine Sand	
3.3	5							Coarse Sand	
5	5.55							Coarse Sand	
5.55	6.5	6	12					Silty Fine Sand	
6.5	10							Silty Fine Sand	
10	11.5	11	21	53					
11.5	15							Silty Fine Sand	
15	16.5							Silty Fine Sand	
16.5	20							S.A.	
20	21.1							Silty Fine Sand	Wet
21.1	21.5	21	19	100				Fine Sandy Clay	
21.5	22							Fine Sandy Clay	
22	25							Silty Fine Sand	
25	25.8						903.48	Silty Fine Sand	
25.8	26.5	26	8					Clay	
26.5	29.5							Clay	
29.5	30							Fine Sand	
30	30.5							Silty Fine Sand	
30.5	31.5	31	13					Coarse Sand	

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		US 177 over Salt Fork River							
Date:		4/11/2012							
Sounding:		B-5							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			914.22						

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	2				Yes	100		Silty Fine Sand	
2	4				Yes	100			
4	6							Silty Fine Sand	
6	7.5	7	3	93				Silty Fine Sand	
7.5	10							Silty Fine Sand	
10	11.1							Silty Sand	
11.1	11.5	11	3	100				Silty Sand	Moist
11.5	12.5						901.72	Silty Sand	
12.5	15							Medium Sand	
15	16.5	16	14	100				Medium Sand	
16.5	20							Medium Sand	Wet starting at 18'
20	21.5	21	9	100				Fine to Medium Sand	Wet
21.5	22.5							Medium Sand	
22.5	24						890.22	Medium to Coarse Sand	
24	25							Medium Sand	
25	26.5	26	2	100				Medium Sand	Super Wet, Hole Caved

Borehole Log								
Project:	ODOT Approach Slab Settlement							
Location:	US 177 over Salt Fork River							
Date:	4/11/2012							
Sounding:	B-6							
Rig Type:								
Drilling Method:		Air Rotary						
Operator:		Larry Taylor (ODOT)						
OU Field Rep.:		Karim Saadedine						
Ground Surface Elev. (ft)		913.08						
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1							Sandy Silt	
1	5							Fine Sand	
5	6.5	6	6	100				Sandy Silt	
6.5	8							Silty Fine Sand	
8	9							Sandy Clay	
9	10							Medium Sand	
10	11.5	11	14	80				Medium Sand	
11.5	12							Medium Sand	
12	13							Sandy Silt	
13	14							Medium Sand	
14	15							Medium Sand	
15	16.2							Silty Sand	
16.2	16.5	16	5	100				Medium Sand	
16.5	17						896.08	Medium Sand	
17	18							Medium Sand with thin layers of Clay	
18	20							Coarse Sand with thin layers of Clay	
20	21.5	21	6					Medium to Coarse Sand	Wet
21.5	23.5							Medium to Coarse Sand	
23.5	23.7							Sandy Clay	
23.7	25							Medium to Coarse Sand	
25	30							Medium to Coarse Sand	
30	35							Medium to Coarse Sand	
35	36.5								No sample, hole caving

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		US 177 over Salt Fork River							
Date:		4/11/2012							
Sounding:		B-7							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			903.36						
							Strata		
Top	Bottom	SPT		SPT		Tube	Change		
Depth	Depth	Depth	SPT	Rec.	Shelby	Rec.	Elev.	Strata	
(ft)	(ft)	(ft)	N-Value	(%)	Tube	(%)	(ft)	Description	Comments
0	4							Silty Sand	
4	5							Silt	
5	5.7	6	10	100				Silty Sand	
5.7	6.5							Fine to Medium Sand	
6.5	10							Fine to Medium Sand	
10	11.5	11	7	93				Fine to Medium Sand	
11.5	12.5							Medium Sand	
12.5	15							Coarse Sand	Water Table at 15 feet
15	16.3	16	7	100			887.06	Medium to Coarse Sand	
16.3	16.5							Clay	
16.5	17.5							Clay	
17.5	19.5				Yes	100			

SH-6 over West Elk Creek

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		SH-6 over West Elk Creek, Elk City							
Date:		7/25/2012							
Sounding:		B-1							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1000.00	Assumed					

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.3						999.70	ACC	
0.03	1.22						998.78	PCC	
1.22	1.65							Coarse to Medium Sand with Trace Clay	
1.65	2.85	2.65	19	80				Coarse to Medium Sand	
2.85	5								
5	6.5	6	6	87					
6.5	10							Coarse to Medium Sand	
10	10.6						989.40	Medium Sand with Trace Clay	
10.6	10.75	11	8	90				Gravel/Sandstone	
10.75	11.35							Reddish Clay	
11.35	15							Reddish Clay	
15	16.5				Yes	75	983.50	Reddish Clay with Crushed Rock	20' Water Level
16.5	20							Reddish Brown Silty Sand	
20	22				Yes	100		Reddish Sandy Silt with Crushed Rock	
22	25							Reddish Brown Sandy Silt	
25	27				Yes	95	973.00	Reddish Brown Sandy Silt with Crushed Rock	
27	30							Reddish Shaley Sandstone	Shelby Tube attempted 30-32', no recovery
30	32	30.2	50 blows per 0.35'	100.000				Reddish Shaley Sandstone	The tube was full of water

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		SH-6 over West Elk Creek, Elk City							
Date:		7/25/2012							
Sounding:		B-2							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1000.00	Assumed					
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	0.09						999.91	ACC	
0.09	1.15						998.85	PCC	
1.15	1.25						998.75	Void	
1.25	1.4						998.60	Stabilized Sand	
1.4	2.9	2.4	11	93				Coarse to Medium Sand	
2.9	5							Coarse to Medium Sand	
5	6.25	6	12	83				Coarse to Medium Sand	
6.25	10							Coarse to Medium Sand	
10	10.5						989.50	Coarse to Medium Sand	
10.5	11.45	11	8	90				Reddish Brown Silty Sand	
11.45	15							Reddish Brown Silty Sand	
15	17				Yes	75		Reddish Brown Sandy Silt	
17	20						980.00	Dark Brown Silty Sand	
20	23				Yes	5		Weathered Sandstone	"crushed sandstone"
23	25				Yes	15		Reddish Brown Silty Sand	
25	26.5	26	9	73				Reddish Dark Brown Silty Sand	Wet

Borehole Log

Project:	ODOT Approach Slab Settlement								
Location:	SH-6 over West Elk Creek, Elk City								
Date:		7/26/2012							
Sounding:	B-3								
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			987.00	Assumed					

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.5						985.50	Reddish Silt	
1.5	2							Dark Brown Sandy Silt	
2	3.5	3	6	87			983.50	Dark Brown Silt	
3.5	5							Reddish Silt with Fine Sand	
5	6.5	6	5	100				Reddish Silt with Fine Sand	
6.5	9						978.00	Reddish Brown Silt with Fine Sand	
9	10							Yellowish Red Silty Medium Sand	
10	11				yes	100		Silty Sand	
11	12							Silty Sand	Wet
12	13	13	6	93			974.00	Reddish Silty Sand	Very Wet
13	13.5							Coarse to Medium Sand with Reddish Sandstone Fragments	
13.5	15							Hard Reddish Sandstone Fragments with Coarse Sand	
15	16.5						970.50	Hard Reddish Sandstone Fragments with Coarse Sand	
16.5	19.5							Sandstone	
19.5	20							Reddish Sandstone, Hard	
20	21.5	21	31+50/0.1	37					

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		SH-6 over West Elk Creek, Elk City							
Date:		7/26/2012							
Sounding:		B-4							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			987.00	Assumed					
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	2						985.00	Brown Silt with Fine Sand	
2	3.5	3	15	100				Silty Sand	
3.5	5							Reddish Silty Sand	
5	6.5	6	2	100				Silty Sand	
6.5	8							Brown Silty Sand	
8	10							Brown Silt with Sand	Wet
10	12							Gravel	Wet
12	13.05	13	3	97				Reddish Silty Sand	
13.05	13.45							Dark Brown Silt	
13.45	15						972.00	Reddish Silty Sand	
15	15.7				yes	35		Silty Sand with Sandstone Fragments	
15.7	16						971.00		
16	18	17	4+50/.27	43				Reddish Shaley Sandstone	
18	19.5	19	10+50/0.27	37				Reddish Shaley Sandstone	

Tecumseh Road over I-35

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		Tecumseh Road over I-35, Norman, OK							
Date:		5/29/2012							
Sounding:		B-2							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1205.34						
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.1						1204.26	PCC	
1.1	1.2						1204.17	Foam	
1.2	1.25						1204.09	Void	
1.3	2							Medium Sand	
2	3.5	3	23	67				Coarse to Medium Sand	
3.5	5							Coarse to Medium Sand	
5	6.5	6	22	87					
6.5	10							Coarse to Medium Sand	
10	11.5	11	6	40					
11.5	12						1193.34	Coarse to Medium Sand	
12	15							Soft Clay	6" of void at 13'
15	17				Yes	75			
17	20							Reddish Clay	
20	22				Yes	75			
22	25							Reddish Clay	Dry
25	25.4						1179.94	Reddish Clay	Water at 25'
25.4	26	26	21	67				Very Dark Greenish Gray Clay	
26	27							Dark Greenish Gray Clay	
27	28							Dark Green Gray Clay with Gravel	
28	29							Reddish Clay	
29	30							Dark Green Gray Clay	
30	32				Yes	100			

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		Tecumseh Road over I-35, Norman, OK							
Date:		5/29/2012							
Sounding:		B-3							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1204.80						
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.2						1203.63	PCC	It rained on 5/28, night
1.2	1.3						1203.51	Void	
1.3	2							Medium Sand	Moist
2.0	2.5						1202.30	Medium Sand	
2.5	5	3	5	0				Reddish Clay	Moist
5	7				Yes	100			
7	10							Reddish Clay	Moist
10	15				Yes	100		Reddish Clay	Dry
15	17				Yes	100			
17	20							Reddish Clay	Moist at 18'
20	24				Yes	95		Reddish Clay	
24	25							Red and Gray Clay	
25	27				Yes	75			
27	28						1176.80	Reddish Clay	
28	30							Very Dark Greenish Gray Clay	
30	32				Yes	100			Water table at 30', Shelby tube was full of water

Borehole Log

Project: ODOT Approach Slab Settlement
Location: Tecumseh Road over I-35, Norman, OK
Date: 5/29/2012
Sounding: B-5
Rig Type:
Drilling Method: Air Rotary
Operator: Larry Taylor (ODOT)
OU Field Rep.: Karim Saadedine

Ground Surface Elev. (ft) 1205.24

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.0						1204.20	PCC	
1.0	1.3						1203.95	Void	
1.3	2							Coarse to Medium Sand	
2.0	2.9	3	6	60				Coarse to Medium Sand	
2.9	5							Coarse to Medium Sand	
5	5.8	6	5	53				Coarse to Medium Sand	
5.8	10							Coarse to Medium Sand	
10	10.9	11	9	63				Coarse to Medium Sand	
10.9	12						1193.24	Coarse to Medium Sand	
12	13						1192.24	Void	
13	17							Reddish Clay	

Borehole Log									
Project:	ODOT Approach Slab Settlement								
Location:	Tecumseh Road over I-35, Norman, OK								
Date:		6/4/2013							
Sounding:	B-9								
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1205.36						

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.0						1204.36	PCC	
1.0	1.3						1204.03	Void/Foam	
1.3	2							Medium Sand	
2.0	2.1	3	4	40			1203.26	Medium Sand	
2.1	2.6							Reddish Clay	
2.6	5							Reddish Clay	
5	10				Yes	65		Reddish Clay	
10	14.5				Yes	100		Reddish Clay	
14.5	15							Reddish Clay with Gravel	
15	15.5				Yes	110		Reddish Clay	
15.5	17							Silty Clay	Dry
17	19							Reddish Clay	
19	20							Clay with Gravel	
20	24.5				Yes	103	1180.86	Reddish Clay	
24.5	25							Dark Greenish Gray Clay	
25	27				Yes	105		Reddish Clay	
27	30							Dark Gray Clay	
30	32				Yes	100			

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		Tecumseh Road over I-35, Norman, OK							
Date:		6/4/2013							
Sounding:		B-11							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1178.36						

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	4.7	1	13	27			1173.66	Brown to Dark Grayish Brown Clay	
4.7	5.0							Reddish Brown Fat Clay	
5.0	8				Yes	110		Reddish Fat Clay	Moist
8	10							Reddish Fat Clay	Dry
10	12				Yes	93		Reddish Fat Clay	Wet
12	15						1163.36	Reddish Fat Clay	Dry
15	15.1				Yes	5		Reddish Shale	
15.1	15.8	16.1	90	40				Reddish Shale	
15.8	21							Reddish Shale	
21	25							White Sandstone	Did not react with HCL

SH-1 over SH-99

Borehole Log

Project:	ODOT Approach Slab Settlement
Location:	SH1 over SH 99 in Ada
Date:	5/15/2012
Sounding:	B-1
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine
Ground Surface Elev. (ft)	1008.80

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.1						1007.70	Concrete	
1.1	1.45						1007.35	Void	
1.45	2.8						1006.00	Stabilized Sand	
2.8	3							Medium Sand	
3	4.5	4	8	93				Medium Sand	
4.5	7							Medium Sand	
7	7.5							Medium Sand	
7.5	10							Medium Sand with Trace Clay	
10	10.2						998.60	Medium Sand with Trace Clay	
10.2	11.5	11	9	100				Reddish Clay	
11.5	13							Reddish Clay with Sand	
13	15				Yes	100		Reddish Clay	
15	18							Reddish Clay	
18	20							Reddish Clay	
20	22				yes	100		Reddish Clay	
22	25							Reddish Clay	Lost Shelby Tube at 25'
25	29							Reddish Clay with Sand	
29	31							Reddish Clay	No penetration because of old Shelby Tube

Borehole Log

Project:	ODOT Approach Slab Settlement		
Location:	SH1 over SH 99 in Ada		
Date:	5/15/2012		
Sounding:	B-2		
Rig Type:			
Drilling Method:		Air Rotary	
Operator:		Larry Taylor	
OU Field Rep.:		Karim Saadedine	
Ground Surface Elev. (ft)		1001.62	

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.1						1000.52	Concrete	
1.1	1.3						1000.32	Void	
1.3	2.5	2.3	40	100			999.12	Stabilized Sand	
2.5	5							Medium Sand	
5	6.5	6	9	100				Medium Sand	
6.5	9						992.62	Medium Sand	
9	10							Clay	
10	12				Yes	65		Clay	
12	15						986.62	Clay	
15	15.3	11	9	100	Yes	15		Green Clay	Shelby Tube Refusal
15.3	15.5							Limestone	
15.5	17							Green Clay	
17	18							Shale with Clay	
18	20							Yellow-Green Clay	
20	22				yes	45			6 TSF to push Shelby Tube

Borehole Log

Project:	ODOT Approach Slab Settlement
Location:	SH1 over SH 99 in Ada
Date:	5/15/2012
Sounding:	B-3
Rig Type:	
Drilling Method:	Air Rotary
Operator:	Larry Taylor (ODOT)
OU Field Rep.:	Karim Saadedine
Ground Surface Elev. (ft)	1001.48

Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.1						1000.38	Concrete	
1.1	2.6	2.3	40	87			998.88	Void	
2.6	5						996.48	Stabilized Sand	
5	6.5	6	10	100				Medium Sand	
6.5	10						991.48	Medium Sand	
10	12				Yes	85			
12	14				Yes	65			
14	16							Green Clay	
16	18						983.48	Soft Sandstone	
18	18.8	11	9	100				Shale	
18.8	19							Reddish Clay	
19	21				Yes	65			

Borehole Log									
Project:		ODOT Approach Slab Settlement							
Location:		SH1 over SH 99 in Ada							
Date:		5/16/2012							
Sounding:		B-4							
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			1008.60						
Top Depth (ft)	Bottom Depth (ft)	SPT Depth (ft)	SPT N-Value	SPT Rec. (%)	Shelby Tube	Tube Rec. (%)	Strata Change Elev. (ft)	Strata Description	Comments
0	1.05						1007.55	Concrete	
1.05	1.3						1007.30	Void	
1.3	2.8	2.3	14	100			1005.80	Stabilized Sand	
2.8	5							Medium Sand	
5	6.5	6	7	100				Medium Sand	
6.5	8							Medium Sand	
8	9.5	9	6	100				Medium Sand	
9.5	13						995.60	Medium Sand	
13	15				Yes	100		Reddish Clay	
15	18							Reddish Clay	
18	20				Yes	35	988.60	Reddish Clay with Sand	Sandstone at 19'
20	22							Reddish Clay	
22	24				Yes	100		Reddish Clay	Some material caved in the Shelby Tube
24	26							Reddish Clay	

Borehole Log									
Project:	ODOT Approach Slab Settlement								
Location:	SH1 over SH 99 in Ada								
Date:		5/23/2012							
Sounding:	B-5								
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			986.23						
							Strata		
Top	Bottom	SPT		SPT		Tube	Change		
Depth	Depth	Depth	SPT	Rec.	Shelby	Rec.	Elev.	Strata	
(ft)	(ft)	(ft)	N-Value	(%)	Tube	(%)	(ft)	Description	Comments
0	2				Yes	70	984.23	Silt with Gravel	
2	2.35						983.88	Silt with Sand	
2.35	2.7						983.53	Silt with Gravel	
2.7	3	3	15	66				Gravel/Limestone	
3	3.8							Gravel	Reacted with HCL
3.8	5.8				Yes	50		Clay	
5.8	6.5							Dry Sandstone	
6.5	8							Moist Sandstone	
8	9.5	8.2	50/5"	0			976.73	Sandstone	
9.5	0	11	9	100				Sandstone	

Borehole Log									
Project:	ODOT Approach Slab Settlement								
Location:	SH1 over SH 99 in Ada								
Date:		5/23/2012							
Sounding:	B-6								
Rig Type:									
Drilling Method:			Air Rotary						
Operator:			Larry Taylor (ODOT)						
OU Field Rep.:			Karim Saadedine						
Ground Surface Elev. (ft)			972.67						
							Strata		
Top	Bottom	SPT		SPT		Tube	Change		
Depth	Depth	Depth	SPT	Rec.	Shelby	Rec.	Elev.	Strata	
(ft)	(ft)	(ft)	N-Value	(%)	Tube	(%)	(ft)	Description	Comments
2.5	3						969.67	Reddish Clay with Gravel	
3	5				Yes	50	967.67	Green Clay	
5	7				Yes	75			
7	7.5								
7.5	9.5				Yes	100			
9.5	10.5							Reddish Clay with Gravel	
10.5	11.5						961.17	Olive Grey Clay	
11.5	12							Limestone	
12	12.5	12	50/1"	37				Limestone	
12.5	20							Limestone	reacted with HCL

Appendix C: Summary of Test Site Soil Properties

SH-59A over Big Creek

Summary of Soil Index and Physical Properties										
Project:	ODOT Approach Slab Settlement									
Location:	SH-59A over Big Creek									
	Ground Surface Elev. (ft)		982.84	BH1						
			982.78	BH2						
			968.41	BH3						
Boring	Depth Range (ft)	Color	Depth (ft)	Elev. (ft)	PL	LL	PI	%fines	wn (%)	LI
BH1	13'-15'	Red	14	968.84	32	56	24	64.5	27.9	-0.18
BH1	20'-22'	Dark Gray	21	961.84	19	38	19	72.2	28.6	0.50
BH1	25'-27'	Gray	26	956.84	21	55	33	80.8	29.2	0.24
BH1	30'-32'	Dark Brown	31	951.84	21	64	43	87.2	32.1	0.26
BH2	10'-12'	Red Brown	11	971.78	35	56	22	81.9	32.3	-0.10
BH2	15'-17'	Red Brown	16	966.78	20	34	14	83.3	29.8	0.67
BH2	20'-21.5'	Dark Gray	20.75	962.03	20	48	28	85.7	---	---
BH2	20.2'-21.5'	Gray	20.75	962.03	22	52	30	---	---	---
BH2	25'-27'	Dark Brown	26	956.78	21	54	33	---	27.1	0.19
BH2	30'-32'	Dark Brown	31	951.78	20	51	31	83.8	30.3	0.34
BH2	35'-37'	Light Brown	36	946.78	30	78	48	---	36.3	0.14
BH3	5.0'-5.5'	Gray	5.3	963.11	21	48	28	81.7	---	---
BH3	15'-17'	---	16	952.41	30	70	41	95.0	29.7	0.00
BH3	20'-22'	---	21	947.41	35	75	40	93.3	33.2	-0.04
BH3	25'-27'	---	26	942.41	35	76	41	93.6	35.1	0.00

US-177 over Salt Fork

Summary of Soil Index and Physical Properties										
Project:	ODOT Approach Slab Settlement									
Location:	SH-59A over Big Creek									
	Ground Surface Elev. (ft)		932.14	A1B1						
			932.12	A1B3						
			929.16	A2B2						
Boring	Depth Range (ft)	Color	Depth (ft)	Elev. (ft)	PL	LL	PI	%fines	wn (%)	LI
A1B1	13'-15'	Brown	14.00	918.14	19	34	16	61.3	22.5	0.25
A1B1	18'-20'	---	19.00	913.14	18	39	21	---	16.7	-0.08
A1B1	20'-23'	---	21.50	910.64	---	---	---	86.3	---	---
A1B1	20'-28'	Brown	21.00	911.14	16	40	24	---	---	---
A1B1	24'-24'8"	Brown	24.50	907.64	16	32	16	80.4	17.0	0.04
A1B1	25'-25'-4"	Light Brown	25.00	907.14	19	40	21	79.1	17.0	-0.09
A1B1	15'-16'	---	15.50	916.64	17	37	20	83.6	---	---
A1B3	15'-17'	---	16.00	916.12	14	26	12	64.6	16.9	0.24
A2B2	14'-14'10"	---	14.50	914.66	---	---	---	79.5	14.5	---
A2B2	14'10"-15'7"	Brown	15.00	914.16	---	---	---	70.9	14.5	---

Tecumseh Road over I-35

Summary of Soil Index and Physical Properties										
Project:	ODOT Approach Slab Settlement									
Location:	Tecumseh Road over I-35, Norman, OK									
	Ground Surface Elev. (ft)		1205.34	BH2						
			1204.80	BH3						
			1205.36	BH9						
			1178.36	BH11						
Boring	Depth Range (ft)	Color	Depth (ft)	Elev. (ft)	PL	LL	PI	%fines	wn (%)	LI
BH1	5'-6'	---	5.5	---	18	46	28	---	---	---
BH2	15'-17'	Red Brown	16	1189.34	12	32	19	68.0	21.3	0.47
BH2	20'-22'	Red Brown	21	1184.34	18	31	13	56.1	17.7	-0.04
BH2	25'-25.4'	Red Brown	25.2	1180.14	---	---	---	68.3	26.5	---
BH3	5'-7'	Red Brown	6	1198.80	18	34	16	84.2	17.6	-0.05
BH3	10'-12'	Red Brown	11	1193.80	19	34	15	85.3	5.1	-0.90
BH3	15'-17'	Red Brown	16	1188.80	17	31	14	60.6	2.4	-1.06
BH3	20'-22'	Red Brown	21	1183.80	18	32	14	60.6	15.7	-0.14
BH3	25'-27'	Brown	26	1178.80	24	36	12	83.2	20.9	-0.25
BH9	2.0'-2.1'	Red Brown	2	1203.36	---	---	---	53.1	---	---
BH9	2.1'-2.6'	Red Brown	2.4	1202.96	19	33	14	72.1	---	---
BH9	5'-7'	Red Brown	6	1199.36	17	34	17	86.5	19.2	0.11
BH9	10'-12'	Red Brown	11	1194.36	19	35	16	83.8	17.0	-0.09
BH9	15'-17'	Red Brown	16	1189.36	18	32	13	64.6	12.8	-0.42
BH9	20'-22'	Red Brown	21	1184.36	20	32	12	67.8	8.6	-0.96
BH9	25'-27'	Red Brown	26	1179.36	17	33	16	72.9	8.5	-0.53
BH9	26'-27'	Brown	26.5	1178.86	16	34	18	82.7	8.5	-0.43
BH11	5'-6'	Red Brown	5.5	1172.86	18	45	28	77.0	8.6	-0.34
BH11	6'-7'	Red Brown	6.5	1171.86	17	44	27	90.3	8.6	-0.30
BH11	10'-12'	Red Brown	11	1167.36	18	33	15	79.2	2.3	-1.04

SH-1 over SH-99

Summary of Soil Index and Physical Properties										
Project:	ODOT Approach Slab Settlement									
Location:	SH-1 over SH-99 in Ada, OK									
	Ground Surface Elev. (ft)		1008.80	BH1						
			1001.62	BH2						
			1001.48	BH3						
			1008.60	BH4						
			986.23	BH5						
			972.67	BH6						
Boring	Depth Range (ft)	Color	Depth (ft)	Elev. (ft)	PL	LL	PI	%fines	wn (%)	LI
BH1	10.2'-11.5'	Gray Brown	10.9	997.90	20	41	20	---	---	---
BH1	13'-15'	Brown	14	994.80	14	30	17	43.4	13.4	-0.01
BH1	20'-22'	---	21	987.80	25	50	25	---	15.3	-0.37
BH2	11'6"-12'	Brown	11.75	989.87	16	33	17	---	---	---
BH2	15'-17'	Gray	16	985.62	15	30	15	52.8	16.1	0.06
BH2	20'-22'	Gray	21	980.62	17	24	7	31.2	8.0	-1.23
BH3	10'-12'	---	11	990.48	14	36	21	---	18.6	0.20
BH3	12'-14'	Red Brown	13	988.48	13	27	14	42.7	14.2	0.08
BH3	19'-21'	Red Brown	20	981.48	22	43	21	85.3	14.6	-0.35
BH4	13'-15'	Brown	14	994.60	---	---	---	44.8	15.1	---
BH4	18'-20'	Light Brown	19	989.60	16	31	15	43.3	17.7	0.14
BH4	22'-24'	Brown	23	985.60	14	33	19	43.2	10.1	-0.21
BH4	24'-26'	---	25	983.60	---	---	---	29.6	8.9	---
BH5	2.35'-2.7'	Brown	2.5	983.73	---	---	---	21.8	---	---
BH5	4'5"-5'1"	Brown	4.75	981.48	24	61	36	---	21.1	-0.09
BH6	5'4"-6'1"	Red Brown	5.75	966.92	22	49	27	85.6	20.0	-0.08
BH6	5'-7'	Red Brown	6	966.67	29	54	25	92.5	20.0	-0.35
BH6	5'-7'	Red Brown	6	966.67	29	52	23	---	20.0	-0.37
BH6	7'6"-7'11"	Brown	7.75	964.92	23	51	29	---	---	---
BH6	7"11"-8'6"	Brown	8.25	964.42	26	52	26	94.2	---	---
BH6	8'6"-9'6"	Red Brown	9	963.67	26	54	28	95.5	---	---
BH6	11'-12'6"	Gray Brown	11.75	960.92	16	33	17	---	---	---