AN OVERVIEW OF MITIGATION STRATEGIES FOR SETTLEMENTS UNDER BRIDGE APPROACH SLABS

by

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ABSTRACT

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Settlement and heave related movements of bridge approach slabs relative to bridge decks create a bump in the roadway causing inconvenience to the travelling public and at times so large as to make travelling unsafe. Hence, it is important to adopt suitable remedial methods to mitigate the approach settlements so as to ensure safe traveling conditions and also to decrease the repair/maintenance costs. This thesis presents an overview of a few case studies on different mitigation techniques applied on bridge approach settlement problems at various locations in the state of Texas. The methods employed to mitigate the settlement including Polyurethane injection, soil nailing, and potential utilization of Geofoam and flowable fill are discussed. Also, the development of design charts for the construction of light weight fill embankments using Expanded Clay and Shale (ECS) aggregate material is presented. Methods on how to use the design charts for various embankments are also covered.

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CHAPTER 1

INTRODUCTION

1.1 General

Bridge approach settlement and the formation of the bump near a bridge are extensively reported (Briaud et al., 1997). This problem usually emanates from soil settlement related problems arising from both embankment fill and subgrade foundation materials. Maintenance of these bridge approach slab settlements cost millions of dollars to repair annually and this mainly absorbs all the maintenance resources. Briaud et al. (1997) reported that 30% of bridges in Texas, i.e., 13,800 out of 46,000 bridges were subjected to the bump problem, while another study cited annual costs for “bump” repairs in Texas is around $7 million (Seo, 2003). These indicate that the bump is a premier maintenance problem in Texas.

There have been several researchers studied the bridge approach settlement to determine both the causes of the bump, and the techniques to mitigate the problem. From the literature review, it is found out that the causes of the bump are various and still too complex to identify. However, the primary sources of the problem can be broadly divided into four categories: 1) Material properties of foundation and embankment, 2) Design criteria for bridge foundation, abutment and deck, 3) Construction method, and 4) Maintenance criteria. It should be noted that not all the factors contribute to the formation of the bump or differential settlement concurrently as one factor may be more problematic than the other. Also, one model cannot be developed for capturing the response of settlements underneath the approach slabs.

For the mitigation techniques, it is found that there have already been several methods employed to alleviate the bump problem, which can be summarized based on the groups of treatments as followings; 1) improvement of foundation soil, 2) improvement of backfill material,

3) design of bridge foundation, 4) design of approach slab, and 5) provide effective drainage and erosion control measures.

One of the major contributors to settlement is the weight of embankment. Hence this research made a comprehensive attempt to address the mitigation technique using Expanded Clay and Shale (ECS) aggregate as a light weight fill material to reduce the weight of the embankment.

The present thesis work primarily focused on the different repair methods including urethane injection, soil nailing and others. Geophysical studies are primarily adopted to identify the settlements under bridge approach slabs of existing bridges and this information is valuable in developing treatment methods. One case study using urethane injection is extensively monitored whereas another with soil nailing is monitored for a short time frame.

Also, charts to design embankments for new bridges using a light weight fill material known as Expanded Clay and Shale (ECS) are introduced. The ECS is a lightweight granular material and is used as a backfill material for embankment construction. With its lightweight property, the ECS fill is expected to decrease the load exerts on the soft foundation soil. As a consequence, the settlement due to consolidation phenomenon will be decreased. .

1.2 Research Objectives

The objectives of this research are to evaluate the effectiveness of different mitigation techniques adapted to repair distressed bridge approach slabs and to develop comprehensive design charts to design bridge embankments using light weight embankment fill material using data from a previous research by Archeewa (2010). Although these methods have already been utilized in other places, they have not been employed in Texas conditions to serve the purpose related to the bump mitigation problem. If this study shows that the different methods employed to repair the distressed bridge approaches of existing bridges and the use of Expanded Clay Shale (ECS) aggregate as a light weight fill material to address the settlement problems in new

bridges, then the results of this study will not only help agencies in lowering their maintenance and repair works of bridge embankments built with clayey soils, but will also reduce traffic congestion problems arisen due to constant repair works.

1.3 Research Report Organization

This thesis consists of five chapters. The first Chapter is an introduction, which presents the background, objectives and tasks involved to accomplish this research.

Chapter 2 presents details of the review from available literature addressing the settlement at bridge approach problem. In the chapter, definition of the bump and the causes of the bump are presented first, and followed by viable techniques used to mitigate the settlement at the bridge approach problem, both for new bridge constructions, and for distressed bridge approach mitigation measures.

Chapter 3 presents the maintenance measures for existing bridges that are primarily remedial methods to repair the settlements under approach slabs. A brief introduction to the need and importance of adopting suitable remedial measures to mitigate the settlement related problems in distressed bridge approach slabs is presented. This chapter provides appropriate mitigation techniques that can be investigated to treat the distressed bridges. Four bridge approach slabs with settlement related problems are identified; the location of the bridges, corrective methods employed, procedure of the method used and the geophysical studies performed on the bridges along with recommendations are discussed.

Chapter 4 presents the study results of a light weight embankment using ECS for new bridge embankment construction. Results discussed in this chapter are from numerical analysis and modeling of the test sections. The results from the laboratory study on ECS that are used as input soil parameters in a finite element numerical model, and the field monitored data of the ECS fill embankment were adopted from previous studies by Archeewa (2010). The comparisons between the results from the FEM modeling and the field data are performed to

validate the numerical model, which are used further to predict the settlements over a long-term condition. Design charts were developed for different heights of embankments, varying subgrade thicknesses and varying compression indices of the subgrade soil.

Summary and conclusions, which include the significant findings from field monitoring and numerical analysis studies, and also limitation of this research and the future needs, are addressed in Chapter 5.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

This chapter presents comprehensive information collected from available literature addressing the problem about the differential settlement at the bridge approach. As a part of this research, the literature review in this chapter was carried out to obtain comprehensive details in five sections. In the first part of this chapter, general information of the definitions of the bump at the end of the bridge and the tolerance of the bump are given. Thereafter, in the second part the various mechanisms causing the formation of the bump such as consolidation of foundation soil, poor compaction of the backfill material, poor water drainage and soil erosion closed to the bridge abutments, types of bridge abutments, traffic volume passing over the bridge decks, age of the approach slab, design of the approach slab, skewness of the bridge and seasonal temperature variations are mentioned. The third part presents the techniques used to mitigate the bump at the end of the bridge for a new bridge. Subsequently, the maintenance measures normally employed by highway agencies to alleviate distressed approach slabs are presented. The final section of the chapter is a summary.

2.2 Definition of Bump and Bump Tolerance

2.2.1 Definition of Bump

Generally, roadway and embankments are built on subgrade foundation and compacted fill materials respectively, which undergo load induced compression and settlements with time. In contrast, the bridges typically need to rest on deep foundations such as pile, pier or other types of deep foundation systems resting on a firm foundation material such as bedrock. Therefore, by resting on a firm foundation the total settlement of the bridge is usually much

smaller than the total settlement of the roadway or adjacent embankment. As a result, a considerable differential settlement occurs at the area between the bridge and roadway interfaces, and a noticeable bump can develop at the bridge ends.

The “Bump” can affect drivers varying from feeling uncomfortable to being hazardous to their lives (Hopkins, 1969; Ardani, 1987). To eliminate the effects of the bump, the approach slab must be built to provide a smooth grade transition between these two structures (bridge and roadway). Another function of the approach slab is to keep the magnitude of differential settlement within a control limit (Mahmood, 1990; Hoppe, 1999). However, in practice it is found that the approach slabs also exceed differential settlements (Mahmood, 1990, Hoppe, 1999). In such cases, the approach slab moves the differential settlement problem at the end of the bridge to the end of the slab connecting with the roadway. Hence, the “Bump” or “Approach Settlement” can be defined as the differential settlement or heave of the approach slab with reference to the bridge abutment structure.

2.2.2 Bump Tolerances

The differential settlement near the bridge approach is a common problem that plagues several bridges in the state of Texas (Jayawikrama et al, 2005). One of the major maintenance problems is to establish severity levels of the bump that require remedial measures. The differential settlement tolerances need to be established for consideration of when to initiate the repair works.

Walkinshaw (1978) suggested that bridges with a differential settlement of 2.5 in. (63 mm) or greater needs to be repaired. Bozozuk (1978) stated that settlement bumps could be allowed up to 3.9 in. (100 mm) in the vertical direction and 2.0 in. (50 mm) in the horizontal direction. Several researchers define the allowable bumps in terms of gradients as a function of the length of the approach slab. Wahls (1990) and Stark et al. (1995) suggested an allowable

settlement gradient as 1/200 of the approach slab length. This critical gradient was also referred by Long et al. (1998), and was used by the Illinois DOT for initiating maintenance operations.

Das et al. (1990) used the International Roughness Index (IRI) to describe the riding quality. The IRI is defined as the accumulations of undulations of a given segment length and is usually reported in m/km or mm/m. The IRI values at the bridge approaches of 3.9 (mm/m) or less indicates a very good riding quality. On the other hand, if the IRI value is equal to 10 or greater, then the approach leading to the bridge is considered as a very poor riding quality. Albajar et al. (2005) established a vertical settlement on the transition zone of 1.6 in. (4 cm) as a threshold value to initiate maintenance procedures on bridge approach areas. In Australia, a differential settlement or change in grade of 0.3% both in the transverse and the longitudinal direction and a residual settlement of 100 mm (for a 40 year design period) are considered as limiting values for bridge approach settlement problems (Hsi and Martin, 2005; Hsi, 2007).

In Texas, the state of practice for repair strategies is different from District to District and these repairs are typically based on visual surveys (Jayawickrama et al., 2005) and International Roughness Index (IRI) values (James et al., 1991). In the study by James et al., (1991), it was indicated that several Districts in Texas have reported bump problems and a few Districts have explored methods such as Urethane injection to moisture control to mitigate settlements. However, these methods have only provided temporary relief as the settlement continues to increase with the service life. As a part of Jayawickrama et al. (2005) study, researchers visited three bridge sites in the Waco, Houston, and San Antonio Districts where Urethane injection was adopted to mitigate approach settlement problems. Their findings are discussed in detail in the subsequent sections of this chapter.

2.3 Mechanisms Causing the Formation of Bump

Bridge approach settlement and the formation of the bump is a common problem that draws significant resources for maintenance, and also creates a negative perception about the

state agencies in the minds of transportation users. From thorough studies compiled from the existing and on-going research studies on the bridge approach settlement, the causes of the problem can be very variable and are still too complex to identify them easily. However, the primary sources of the problem can be broadly divided into four categories; material properties of foundation and embankment, design criteria for bridge foundation, abutment and deck, construction supervision of the structures, and maintenance criteria. It should be noted that not all the factors contribute to the formation of the bump concurrently.

There have been many studies employed across the states in the USA to study the causes of the problem and the methodologies to solve it (Hopkins, 1969, 1985; Stewart, 1985; Greimann et al., 1987; Laguros et al, 1990; Kramer and Sajer, 1991; Ha et al, 2002; Jayawikrama et al, 2005; White et al, 2005, 2007).

White et al. (2005) define the term “bridge approach,” not just in terms of the approach slab alone, but in terms of a larger area, covering from the bridge structure (abutment) to a distance of about 100 ft away from the abutment. This definition includes the backfill and embankment areas under and beyond the approach slab as significant contributors to the settlements in the bridge approach region.

Many factors are reported in the literatures that explain the mechanisms causing the formation of bumps on the bridge transition (Hopkins, 1969; Stewart, 1985; Kramer and Sajer, 1991). According to Hopkins (1969), the factors causing differential settlement of the bridge approaches are listed as:

a. Type and compressibility of the soil or fill material used in the embankment and foundation
b. Thickness of the compressible foundation soil layer
c. Height of the embankment
d. Type of abutment.

Kramer and Sajer (1991) and Briaud et al. (1997) concurred with these observations later based on extensive surveys of various State DOTs in the USA. Stewart (1985) performed a research study for Caltrans and this study concurred with the finding reported by Hopkins (1969), in particular the observations noting that the original ground and fill materials contribute the maximum settlement to the approach slab. Based on the results obtained from a field study performed at Nebraska, Tadros and Benak (1989) confirmed that the primary cause of this problem is due to the consolidation of foundation soil but not the consolidation of the compacted embankment fill. The proper compaction of the embankment in accordance with the construction specifications has an important influence on the settlement of embankment fill material. Also, the swell and shrink behaviors of the foundation/ backfill soil and vibration or movements of the backfill soil (in case of granular fill) due to moving traffic loads may significantly impact the development of the approach faults (Hopkins, 1969, 1985).

Ardani (1987), Wahls (1990), and Jayawikrama et al. (2005) also reported that both the time-dependent settlement (primary/secondary consolidation) of foundation soil beneath the embankment and the approach slab embankment as well as the poor compaction of embankment adjacent to the abutment, and erosion of soil at the abutment face and poor drainage system around the abutment are the major contributors to approach settlement problems.

Wahls (1990) stated that the approach-slab design and the type of abutment and foundation can affect the relative settlement of the slab and bridge abutment. Abutments supported by the shallow foundations and when these foundations lay within the approach embankment fill will settle along with the embankment. In addition, Wahls (1990) concluded that the lateral creep of foundation soils and lateral movement of abutments can potentially cause this problem.

Laguros et al. (1990) reported that factors including the age of the approach slab, height of embankment, skewness of the bridge and traffic volume influence the bridge approach settlement. The flexibility of the approach pavements has a considerable influence as well. Laguros et al. (1990) observed greater differential settlement in flexible pavements than rigid pavements during initial stages following construction (short term performance), while both pavement types performed similarly over the long term. More details are provided in later sections.

Other factors that influence the creation of the bump include the type of bridge abutment and approach slab design (Mahmood, 1990; Wahls, 1990). Design of abutment structures is not unique and varies as per the connection of the slab with the abutment. The abutments are characterized as mainly integral (movable) or non-integral (conventional or stub) type of abutments (Greimann et al., 1987). For an integral abutment, the bridge deck slab is monolithically connected to the abutment, and the abutment is allowed to move laterally along with the bridge deck slab; while for a non-integral one, the bridge deck is independent of the abutment, and the longitudinal movements of the bridge deck are taken care of by roller/pin-bearing plates.

Weather changes also contribute to the differential settlement between the bridge and the approach slab as in the case of integral abutments when seasonal temperature changes from summer to winter (Schaefer and Koch, 1992). Weather changes often lead to soil displacement behind the abutment eventually leading to void development under the approach slab (Schaefer and Koch, 1992; White et al., 2005). This creates water infiltration under the slab, which leads to erosion and loss of backfill material (Jayawikrama et al., 2005).

White et al. (2007) carried a comprehensive field study of 74 bridges in Iowa to characterize problems leading to poor performance of bridge approach pavement systems. White et al. (2007) claimed that subsurface void development caused by water infiltration

through unsealed expansion joints; collapse and erosion of the granular backfill, and poor construction practices were found to be the main contributing factors of the approach slab settlements in Iowa.

Other research studies from outside the USA, including Australia and China show that the bump at the end of the bridge is a major concern in highway and freeway constructions. Hsi and Martin (2005) and Hsi (2007) reported that the approach settlement problems were observed due to very soft estuarine and marine clays in subsoils at the construction of the Yelgun-Chinderah Freeway in New South Wales, Australia. Hsi (2007) reported that rapid construction of deep approach embankments over very soft clay subgrades often experienced the long term settlement of the soft subgrade which has attributed to causing settlements at the approach slabs.

In the following, three studies by Briaud et al. (1997), Seo (2003) and White et al. (2007) listed factors that contribute to bumps. Briaud et al. (1997) summarized various factors that contributed to the formation of bumps/settlements at the approach slabs in Figure 2.1. These factors were grouped and ranked in the following order in which they contribute to the soil movements: fill on compressible foundation; approach slab too short; poor fill material; compressible fill; high deep embankment; poor drainage; soil erosion; and poor joint design and maintenance.

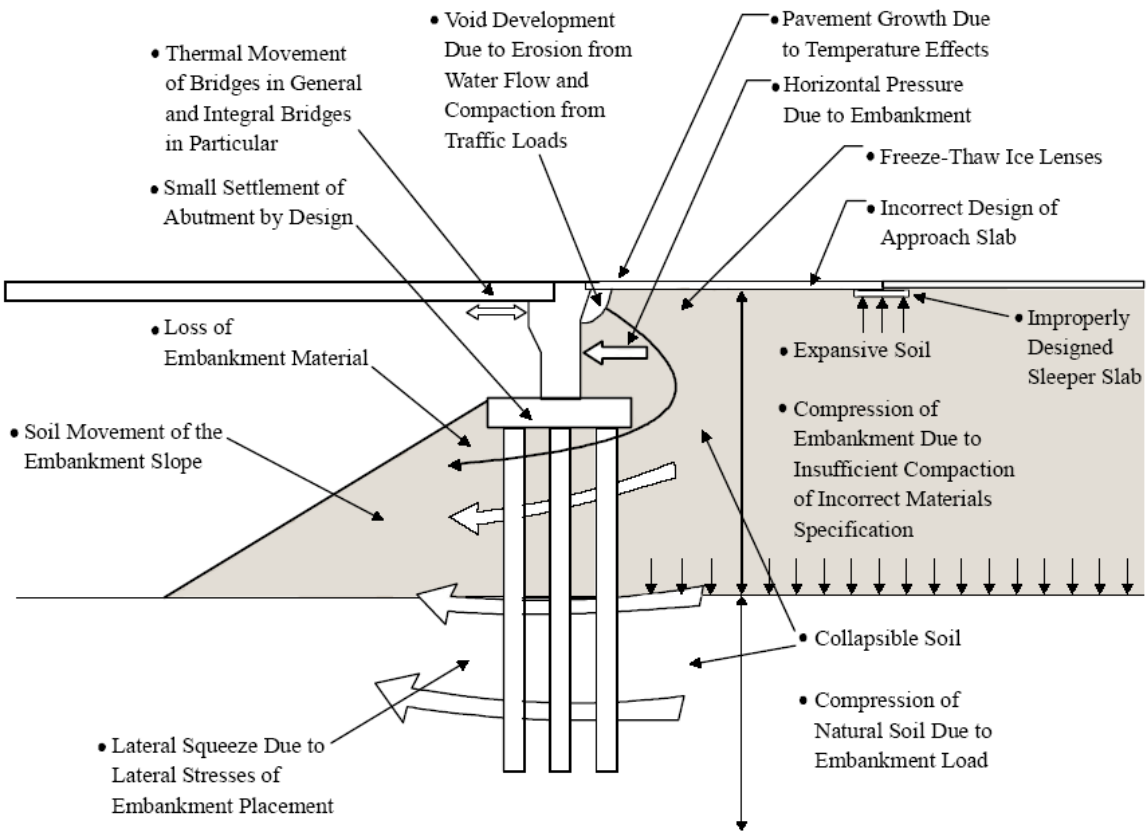


Figure 2.1 Schematic of different origins leading to formation of bump at the end of the bridge (Briaud et al., 1997)

Seo (2003) performed a circular track test involving the approach slab which was repeatedly loaded by a vehicle model. Seo (2003) listed the following observations:

1. Number of cycles of loading over the approach slab is proportional to the increase in the bump
2. Shorter approach slabs result in higher displacements of the slab
3. More highly compacted stiffer soils result in less deflection of the slab
4. The velocity of vehicles has an influence on the increase in magnitude of the bump
5. The weight of vehicles relates to the degree of the settlement.

A recent study conducted by White et al. (2007) summarized the following factors as contributors to differential settlements of the approach slab:

1. Backfill materials under poorly performing approach slabs are often loose and under compacted.
2. The foundation soil or embankment fill settles.
3. Many bridge approach elevation profiles have slopes higher than 1/200, which is considered a maximum acceptable gradient for bridge approaches.
4. Voids develop under bridge approaches within one year of construction, indicating insufficiently compacted and erodible backfill material.
5. Inadequate drainage is a major bridge approach problem. Many abutment subdrains are dry with no evidence of water, are blocked with soil and debris, or have collapsed.
6. Many expansion joints are not sufficiently filled, allowing water to flow into the underlying fill materials.

By summarizing the above studies as well as a review of other investigations that addressed this bump problem, the major factors that cause approach bumps can be listed as:

1. Consolidation settlement of foundation soil
2. Poor compaction and consolidation of backfill material
3. Poor drainage and soil erosion
4. Types of bridge abutments
5. Traffic volume
6. Age of the approach slab
7. Approach slab design
8. Skewness of the bridge
9. Seasonal temperature variations

2.4 Mitigation Techniques for Approach Settlements of New Bridges

This sub-section is a summary of methods adopted for mitigating potential settlements expected in new bridges. The treatment techniques adopted to mitigate the settlements can be classified based on their area of application such as

1. Improvement of foundation soil
2. Improvement of backfill material
3. Design of bridge foundation
4. Design of approach slab
5. Effective drainage and erosion control methods

The objective of this research is to establish the use of light weight fill material to reduce the settlements under approach slab. Hence the treatment techniques adopted to improve the backfill material are considered.

The bridge approach embankment has two functions; first to support the highway pavement system, and second to connect the main road with the bridge deck. Most of the approach embankments are normally constructed by conventional compaction procedures using materials from nearby roadway excavation or a convenient borrow pit close to the bridge site. This implies that the serviceability of the embankment, in the aspects of slope stability, settlement, consolidation, or bearing capacity issues, depends on the geotechnical properties of the fill materials (Wahls, 1990). In addition, since the embankment must provide a good transition between the roadway and the bridge, the standards for design and construction considerations both in materials quality requirements and compaction specifications must be specified in order to limit the settlement magnitude within a small acceptable degree (Wahls, 1990).

Generally, the materials for embankment construction should have the following properties (White, 2005):

a. being easily compacted,
b. not time-dependent,
c. not sensitive to moisture,
d. providing good drainage,
e. erosion resistance and
f. shear resistance.

Dupont and Allen (2002) cited that the most successful method to construct the approach embankments is to select high quality fill material, with the majority of them being a coarse granular material with high internal frictional characteristics. Several research methods have been attempted to define methods to minimize potential of settlement and lateral movement development in the approach embankments and these studies are discussed in the following.

Hoppe (1999) studied the embankment material specifications from various DOTs. The results from his survey are presented in Tables 2.1 and 2.2. It can be seen from Table 3 that forty-nine (49) percent of the state agencies use more rigorous material specifications for an approach fill than for a regular highway embankment fill. Furthermore, the study also shows that typical requirements for the backfill materials among the different states varied with one another. One common requirement followed by several states is to limit the percentage of fine particles in the fill material in order to reduce the material plasticity. As an example, the allowable percentage of material passing the No. 200 (75-micron) sieve varies from less than 4% to less than 20%. Another requirement commonly found is to enhance the fill drainage properties by a requisite of pervious granular material.

Table 2.1 Embankment Material Specifications (Hoppe, 1999)

State	Same/Different from Regular Embankment	% Passing 75 mm (No.200 sieve)	Miscellaneous
AL	Same		A-1 to A-7
AZ	Different		
CA		<4	Compacted pervious material
CT	Different	<5	Pervious material
DE	Different		Borrow type C
FL	Same		A-1, A-2-4 through A-2-7, A-4, A-5, A-6, A-7 (LL<50)
GA	Same		GA Class I, II or III
ID			A yielding material
IL	Different		Porous, granular
IN	Different	<8	
IO	Different		Granular; can use Geogrid
KS			Can use granular, flowable or light weight
KY		<10	Granular
LA			Granular
ME	Different	<20	Granular borrow
MA	Different	<10	Gravel borrow type B, M1.03.0
MI	Different	<7	Only top 0.9 m (3 ft) are different (granular material Class II)
MN		<10	Fairly clean granular
MO			Approved material
MS	Different		Sandy or loamy, non-plastic
MT	Different	<4	Pervious
NE			Granular
NV	Different		Granular
NH	Same	<12	
NJ	Different	<8	Porous fill (Soil Aggregate I-9)
NM	Same		
NY		<15	<30% Magnesium Sulfate loss
ND	Different		Graded mix of gravel and sand
OH	Same		Can use granular material
OK	Different		Granular just next to backwall
OR	Different		Better material
SC	Same		
SD	Varies		Different for integral; same for conventional
TX	Same		
VT	Same		Granular
VA	Same		Pervious backfill
WA			Gravel borrow
WI	Different	<15	Granular
WY	Different		Fabric reinforced

Table 2.2 Lift Thickness and Percent Compaction Requirements (Hoppe, 1999)

State	Lift Thickness, mm(in.)	% Compaction	Miscellaneous
AL	203(8)	95	
AZ	203(8)	100	
CA	203(8)	95	For top 0.76 m (2.5 ft)
CT	152(6)	100	Compacted lift indicated
DE	203(8)	95	
FL	203(8)	100	
GA		100	
ID	203(8)	95	
IL	203(8)	95	For top, remainder varies with embankment height
IN	203(8)	95	
IO	203(8)	None	One roller pass per inch thickness
KS	203(8)	90	
KY	152(6)	95	Compacted lift indicated; Moisture = +2% or -4% of optimum
LA	305(12)	95	
ME	203(8)		At or near optimum moisture
MD	152(6)	97	For top 0.30 m (1ft), remainder is 92%
MA	152(6)	95	
MI	230(9)	95	
MN	203(8)	95	
MO	203(8)	95	
MS	203(8)		
MT	152(6)	95	At or near optimum moisture
NE		95	
NV		95	
NH	305(12)	98	
NJ	305(12)	95	
NY	152(6)	95	Compacted lift indicated
ND	152(6)		
OH	152(6)		
OK	152(6)	95	
OR	203(8)	95	For top 0.91 m (3ft), remainder is 90%
SC	203(8)	95	
SD	203-305(8-12)	97	0.20 m (8 in.) for embankment, 0.30 m (12 in.) for bridge end backfill
TX	305(12)	None	
VT	203(8)	90	
VA	203(8)	95	+ or – 20% of optimum moisture
WA	102(4)	95	Top 0.61 m (2 ft), remainder is 0.20 m (8 in.)
WI	203(8)	95	Top 1.82 m (6 ft and within 60 m (200 ft) remainder is 90%
WY	305(12)		Use reinforced geotextiles layers

From the same study by Hoppe (1999), two other conclusions can be further drawn from Table 2.2 above. First, in many states, a 95% of the standard proctor test compaction condition is generally specified for the compaction of approach fill.

Second, the approach fill material is normally constructed at a lift thickness of 8 in. In Texas, a loose thickness of 12 in. compacted to 8 in. of fill is commonly used and the percent compaction is not always specified. Dupont and Allen (2002) also conducted another survey of 50 state highway agencies in the USA in order to identify the most common type of backfill material used in the embankments near bridge approaches. Their study shows that most of the state agencies, i.e. 38 states use granular material as the backfill; 3 states use sands; 6 states use flowable fill; while 17 states use compacted soil in the abutment area.

A few other research studies were conducted to study the limitations of the percent fine material used in the embankment fill. Wahls (1990) recommended that the fill materials should have a plasticity index (PI) less than 15 with percent fines not more than 5%. The FHWA (2000) recommended backfill materials with less than 15% passing the No. 200 sieve. Another recommendation of the backfill material by Seo (2003) specifies the use of a backfill material with a plasticity index (PI) less than 15, with less than 20 percent passing the No. 200 sieve and with a coefficient of uniformity greater than 3. This fill material is recommended to be used within 100 ft of the abutment.

For the density requirements, Wahls (1990) suggested two required density values; one for roadway embankments and the other for bridge approaches. For embankment material, the recommended compaction density is 90 to 95 percent of maximum dry density from the AASHTO T-99 test method, while the density for the bridge approach fill material is recommended from 95 to 100 percent of maximum dry density from the AASHTO T-99 test method. Wahls (1990) also stated that well-graded materials with less than 5% passing the

No. 200 sieve are easy to be compacted and such material can minimize post construction compression of the backfill and can eliminate frost heave problems.

Seo (2003) suggested that the embankment and the backfill materials within the 100 foot-length from the abutment should be compacted to 95% density of the modified proctor test. White et al. (2005) also recommended the same compaction of 95% of the modified proctor density for the backfill. White et al. (2005) also used a Collapse Index (CI) as a parameter to identify an adequacy of the backfill material in their studies. The CI is an index, which measures the change in soil volume as a function of placement water content. It was found that materials placed at moisture contents in the bulking range from 3% to 7% with a CI value up to 6% meet the Iowa DOT specifications for granular backfills.

In the current TxDOT Bridge Design Manual (2001), the approach slab should be supported by the abutment backwall and the approach backfill. Therefore, the backfill materials become a very important aspect in an approach embankment construction. As a result, the placement of a Cement Stabilized Sand (CSS) “wedge” in the zone behind the abutment is currently practiced by TxDOT. The placement of the CSS “wedge” in the zone behind the abutment is to solve the problems experienced while compacting the fill material right behind the abutment. This placement also provides a resistance to the moisture gain and loss of material, which are commonly experienced under approach slabs. The use of CSS has become standard practice in several Districts and has shown good results according to the TxDOT manual.

Apart from the embankment backfill material and construction specifications, the other alternatives, such as using flowable fills (low strength and flowable concrete mixes) as backfill around the abutment, wrapping layers of backfill material with Geosynthetic or grouting have also been employed to solve the problem of the excessive settlements induced by the embankment. The use of these construction materials and new techniques increases

construction costs inevitably. However, the increased costs can be balanced by the benefits obtained by less settlement problems. For example, the use of Geosynthetic can prevent infiltration of backfill into the natural soil, resistance against lateral movements and improves the quality of the embankment (Burke, 1987).

Another concept to reduce the vertical loading or stress exerted by the embankment on the foundation subsoil is the use of lightweight material as an embankment fill material. The reduction of embankment weight or load increases the stabilities of the embankment and also reduces the compression on the underlying foundation soil. As a result, the settlement potential of the embankment will be decreased.

The lightweight fills such as lightweight aggregate, expanded polystyrene, lightweight concrete, or others can be used to achieve this benefit (Luna et al., 2004, Dupont and Allen, 2002, Mahmood, 1990). Based on the surveys conducted by Hoppe (1999) approximately 27% of responding DOTs have already experimented with the use of non-soil materials behind bridge abutments.

Horvath (2000) recommended the use of Geofoam as a light weight compressible fill material (Figure 2.2). Other materials could be used as alternative lightweight backfill material; some of these alternative construction materials included shredded tires and expanded polystyrene. However, it must be kept in mind that the suitable fill material must not have only the lightweight property, but it must have other required properties, such as, high strength, high stiffness and low compressibility properties.

Hartlen (1985) listed some satisfactory requirements for the lightweight fill material as follows;

a. Bulk density less than 63 pcf. (1000 kg/m3)
b. High modulus of elasticity and high angle of internal friction
c. Good stability and resistance against crushing and chemical deterioration

d. Non-frost active

e. Non-corrosive to concrete and steel

f. Non-hazardous to the environment

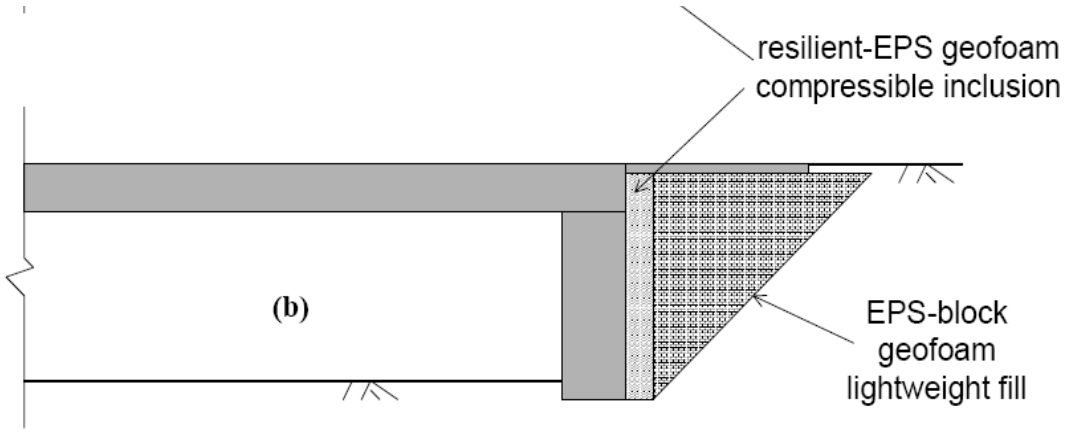


Figure 2.2 A design alternative by using geofoam as a backfill (Horvath, 2000)

2.5 Maintenance Measures For Distressed Approach Slabs

This subchapter presents several techniques normally used to treat distressed approach slabs. It is estimated that bridge approach maintenance costs are at least $100 million per year in the United States (Briaud et al, 1997; Nassif et al., 2002). Many states indicate that the best practice to minimize the presence of bridge bumps is to establish up-to-date maintenance activities, by scheduling periodic repair activities in addition to occasional required maintenance (Dupont and Allen, 2002). Depending on the circumstances, maintenance of distressed approach slabs is comprised of asphalt overlays, slab jacking, and approach slab adjustment or replacement techniques (Dupont and Allen, 2002).

It is also reported that in the case of conventional bridges, much of the cost of maintenance is related to repair of damage at joints, because such joints require periodic

cleaning and replacement (Briaud, 1997, Arsoy, 1999). Other times, pavement patching at the ends of the bridge represents most of the maintenance costs. For longer bridges, the pavement patching lengths are longer due to problems experienced by the temperature induced cyclic movements (Hoppe, 1999). However, Arsoy (1999) noted that Integral abutment bridges perform well with fewer maintenance problems than conventional bridges.

Also, a periodic cleanout and maintenance schedule is required for all drainage structures on the bridge and bridge approach system to insure proper removal of water away from the structure and to minimize runoff infiltration into underlying fill layers (Lenke, 2006). Most frequently, maintenance of drainage structures and joints is lacking and must be improved in order to take full advantage of these design features (Lenke, 2006, Wu et al., 2006).

Lenke (2006) presented his study showing many cases of poor maintenance at the expansion joints between the bridge deck, approach slab, and approach pavement, and drainage systems, resulting in many bridge replacement and rehabilitation costs. He suggested that to prevent stress buildup at the expansion joints between the bridge structure, the approach slab and the pavement system, a good maintenance by cleaning and replacement (when necessary) is required. Such stresses can not only cause damage to the deck and the abutment, but can also cause distortions of the approach slab.

Lenke (2006) also identified another maintenance issue resulting from Alkali-Silica Reactivity (ASR) problems. The stresses caused by ASR expansion can lead to severe damage at the joints connecting the bridge deck to the approach slab and the approach slab to the preceding concrete pavement. These ASR expansion stresses can cause spalling and resultant crack widening, which regularly requires joint filling with bituminous materials work (Lenke, 2006).

White et al. (2005) also conducted a comprehensive study in a case of lack of maintenance of drainage structures, such as clogged or blocked drains, animal interaction, and

deterioration of joint fillers, gutters and channels. The study showed that due to the lack of maintenance many problems about maintenance occurred, resulting in numerous and costly repair operations. White et al. (2005) also pointed out some potential causes of bridge approach settlement discovered during the maintenance activities. For example, they mentioned that the loose and not properly compacted backfill materials can cause poorly performing approach slabs. Coring operations revealed that voids are highest near the bridge abutment and decreased with distance with void sizes ranging from 0.5 in. to 12 in. Snake cameras used at sub-drain outlets demonstrated that most of the investigated sub drains were not functioning properly. The sub drains were either dry with no evidence of water or blocked with soil fines and debris or had collapsed. Some of these problems are attributed to erosion induced movements in the fill material from moisture infiltration. This signifies the need for constant maintenance of joints and drains so that infiltration into the soil layers will be low. Along with the maintenance, reconstruction or rehabilitation of distressed approach slabs are very necessary.

Several soil stabilization techniques were found in the literature to stabilize the fill under the approach slab. These techniques are intended to smooth the approaches by raising the sleeper slab and approaches, especially if application of an asphalt overlay is not feasible (Abu-Hejleh et al., 2006). The most important techniques are pressure grouting under the slab, slab-jacking or mud-jacking technique, the Urethane method, and compaction or high pressure grouting. Most of these techniques are often used as remedial measures after problems are detected. However, the same could be applied even in new bridge constructions. A brief overview of these methods is presented below.

2.5.1 Replacement Method

Highly deteriorated approach slabs due to the formation of a bump are mostly replaced with the new approach slabs. This process is the most expensive and time taking process as

the construction process results in frequent closure of lanes, traffic congestion, etc. A new internal research project has been initiated by the California Department of Transportation to examine different replacement alternatives for deteriorated approach slabs. In this project, prefabricated Fiber Reinforced Polymer (FRP) decks as well as FRP gridforms and rebars were investigated as replacement options. Full scale approach slabs were tested under simulated wheel loads. Performance of the approach slabs were also examined under simulated washout conditions. Figure 2.3 shows the test schematic.

Figure 2.3 Simulated approach slab deflection due to washout by UC Davis research team (http://cee.engr.ucdavis.edu/faculty/chai/Research/ApproachSlab/ApproachSlab.html)

2.5.2 Mud/Slab Jacking

Mud/Slab jacking is a quick and economical technique of raising a settled slab section to a desired elevation by pressure injecting of cement grout or mud-cement mixtures under the slabs (EM 1110-2-3506, 20 Jan 84). According to EM 1110-2-3506, slab jacking is used to improve the riding qualities of the surface of the pavement, prevent impact loading over the

irregularities by fast-moving traffic, correct faulty drainage, prevent pumping at transverse joints, lift or level other structures, and prevent additional settlement.

In this method, the mud grout is prepared using the topsoil which is free from roots, rocks and debris mixed with cement and enough water to produce a thick grout. This grout is injected to fill the void spaces underneath the approach slab through grout holes made through the approach slabs (Bowders et al, 2002). The injection is performed in a systematic manner to avoid cracks on the approach slab as shown in Figure 2.4. Precautionary measures need to be taken near to side retaining walls and abutment walls (Luna et al., 2004).

Even though this technique has been successfully adopted by several states including Kentucky, Missouri, Minnesota, North Dakota, Oklahoma, Oregon, and Texas for lifting the settled approach slabs, the mud/slab jacking can be quite expensive. Mud jacking may also cause drainage systems next to the abutment to become clogged, and is difficult sometimes to control the placement of the material (Dupont and Allen, 2002). Other difficulties including limited spread of grout into voids, large access holes which must be filled and lack of sufficient procedural process made this technique as uneconomical (Soltesz, 2002). Abu al-Eis and LaBarca, (2007) reported that the cost of this technique was between $40 and $60 per one square yard of pavement used based on two test sections constructed in Columbia and Dane counties in Wisconsin.

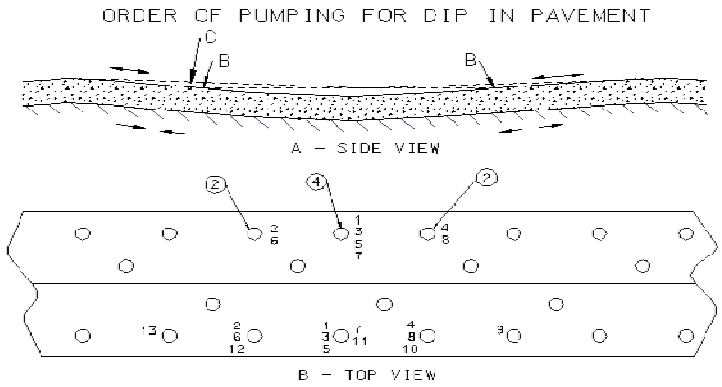


Figure 2.4 Mud-jacking injection sequences (MoDOT, EPC)

2.5.3 Grouting

2.5.3.1 Pressure grouting under the slab

The presence of voids beneath the approach slab can lead to instability, cracking, sinking and pounding problems (Abu-Hejleh et al., 2006). In order to mitigate the problem, pressure grouting is commonly used for bridge approach maintenance practice as a preventive measure (White et al., 2005 and 2007). Pressure grouting under the slab is used to fill the voids beneath the approach slab through injection of flowable grout, without raising the slab (Abu-Hejleh et al., 2006).

According to White et al. (2007), under sealing the approach slab by pressure grouting normally has two operations within the first year after completion of approach pavement construction. The first operation is done within the first 2–6 months, while the second one is employed within 6 months after the first under sealing. The grout mix design consists of Type 1 Portland cement and Class C fly ash at a ratio of 1:3. Water is also added in the grouting material to achieve the specified fluidity (Buss, 1989). Moreover, in order to avoid the lifting of the approach slab, grout injection pressures are kept to less than 35 kPa (White et al., 2007).

Abu-Hejleh et al. (2006) stated that the construction techniques for this method are to drill 1-7/8" holes through the concrete or asphalt approach slabs using a rectangular spacing as shown in Figure 2.5. The depth is determined by the ease of driving the stinger or outlet tube, which is pounded into the hole (Abu-Hejleh et al., 2006). A fence post pounder is used to hammer the stinger and extension pieces into the soil (Abu-Hejleh et al., 2006). As the stinger is pounded down, the operator can determine if the soil is loose or soft and if there are voids under the slab.

Although grouting under the approach slab is commonly used for bridge approach settlement as a mitigation method, White et al. (2007) stated that the grouting is not a long term solution for this problem. The grouting does not prevent further settlement or loss of backfill material due to erosion (White et al., 2005 and 2007).

2.5.3.2 Compaction or High Pressure Grouting

Compaction grouting is a method for improving soil by densifying loose and liquefaction soils and resulting in increasing the soil strength (Miller and Roykroft, 2004). The compaction grouting is a physical process, involving pressure-displacement of soils with stiff, low-mobility sand-cement grout (Strauss et al., 2004).

According to the ASCE Grouting Committee (1980), the grout generally does not enter the soil pores but remains as a homogenous mass that gives controlled displacement to compact loose soils, gives controlled displacement for lifting of structures, or both. The FHWA (1998) also stated that apart from soil densification, the compaction grouting is also employed to lift and level the approach slab and adjacent roadways.

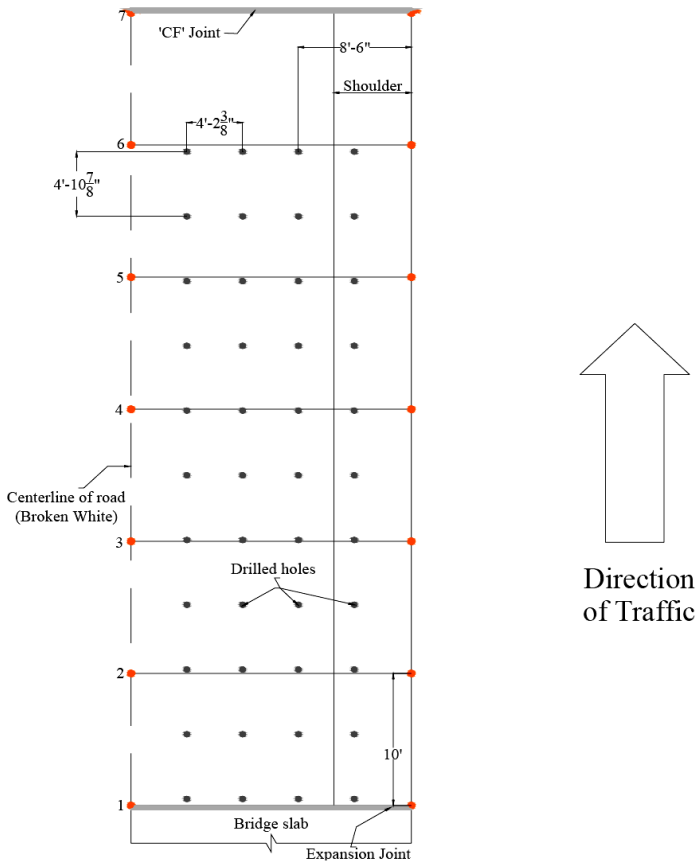


Figure 2.5 Location of holes drilled on an approach slab (White et al., 2005)

The compaction grouting can be used to stabilize both shallow and deep seated soft layers (Abu-Hejleh et al., 2006). Section 211 of the CDOT Standard Specifications describes the grouting must be low slump and a low mobility grout with a high internal friction angle. When the technique is used in weak or loose soils, the grout typically forms a coherent “bulb” at the tip of the injection pipe; thus, the surrounding soil is compacted and/or densified (Miller and Roykroft, 2004). For relatively free draining soils including gravel, sands, and coarse silts the method has proven to be effective (Abu-Hejleh et al., 2006).

2.5.3.3 Urethane Injection Technique

The Urethane injection technique was first developed in 1975 in Finland to lift and under seal concrete pavements and subsequently adopted in several US States in lifting concrete pavements (Abu al-Eis and LaBarca, 2007). In this process, a resin manufactured from high density polyurethane is injected through grout holes (5/8 inch diameter) made through the approach slab to lift, fill the voids and to under seal the slab (Abu al-Eis and LaBarca, 2007). The injected resin will gain 90% of its maximum compressive strength (minimum compressive strength is 40 psi) within 15 minutes. Once the voids are filled, the grout holes are filled with inexpansive grout material. Elevation levels are taken before and after the process to ensure the required lifting is achieved (Abu al-Eis and LaBarca, 2007).

As reported by Abu al-Eis and LaBarca, (2007), the Louisiana Department of Transportation successfully adopted this technique for two different bridge approaches and observed that the international roughness index (IRI) values were reduced by 33% to 57% after monitoring for four years. This method involves the precise liquid injection of high-density polyurethane plastic through small (5/8") holes drilled in the sagging concrete slab (Abu al-Eis and LaBarca, 2007). Once it is applied, the material expands to lift and stabilize the slab, while filling voids in the underlying soil and under sealing the existing concrete (Concrete Stabilize Technology Inc.,http://www.stableconcrete.com/uretek.html). Based on the manufacturer provided information, this technology is simple and rapid. It can lead to a permanent solution and also can resist erosion and compression over a time period.

Brewer et al., (1994) first evaluated the Urethane injection technique to raise bridge approach slabs in Oklahoma. They reported that three test slabs out of six were cracked during or after the injection and in one case, the PCC slab broke in half during the injection. The Michigan Department of Transportation reported that this technique provided temporary increase in base stability and improvement in ride quality for one year (Opland and Barnhart, 1995).

Soltesz (2002) noticed that the Urethane treatment was successful even after two years where the injection holes are properly sealed. The Oregon Department of Transportation researchers reported that the Urethane material was able to penetrate holes with diameters as small as 1/8 in. and which was added advantage of this technique to fill the minor pores of the subbase and lift the pavement slabs (Soltesz, 2002).

Abu al-Eis and LaBarca, (2007) reported that the cost of this technique was between $6 to $7 per pound of foam used which was calculated based on two test sections constructed in Columbia and Dane counties in Wisconsin. They summarized the cost comparison of this technique with other slab lifting methods (as shown in Table 2.3) and concluded that this technique is expensive when compared to other methods if calculated based on direct costs. They also reported that this technique is very fast and can open the lanes for traffic immediately after the treatment. The amount of urethane resin used in each project is also questionable as this quantity is directly used in the cost analysis. Considering this fact, TXDOT amended its Special Specification 3043-001 which requires a Special Provision for determining the quantity of polymer resin used for “Raising and Undersealing Concrete Slabs”. Regarding the Special Specification 3043-001, the quantity of the resin utilized will be calculated by one of the following methods:

Payment will be made according to the actual quantity of polymer resin used in the work by determining the weight of material placed by measuring the depth of polymer resin in the holding tanks before and after each day’s work. A Professional Engineer and a site engineer must approve the calculation method which is based on the certified measured volume of each tank and the unit weight of each component to determine the weight of resins used in the work.

Table 2.3 Cost Comparison for Four Slab Faulting Repair Methods (Abu al-Eis and LaBarca, 2007)

Location	Method	Total Cost	Cost per yd^2	Days to Complete
I-30 (80 yd^2)	URETEK	$19,440	$243	0.75
	Slab Replacement	$34,000	$425	3
	HMA Overlay	$3,630	$45	1
	Mud-jacking	$3,000	$38	1
USH 14 (53.4 yd^2)	URETEK	$6,260	$117	0.5
	Slab Replacement	$22,670	$425	3
	HMA Overlay	$3,375	$63	1
	Mud-jacking	$3,000	$56	1

Several Districts in Texas use this method as a remediation method and based on the present research contacts, these methods are deemed effective. Several Houston sites visited were repaired utilizing this injection method ten years ago and they are still functioning adequately. The work reported in the Houston District was instrumental in the development of the TxDOT Special Specification for the use of the urethane injection method for lifting the distressed approach slabs.

2.5.3.4 Flowable fill

Flowable fill or controlled low-strength material is defined by ACI Committee 229 as a self-compacting, cementitious material used primarily as a backfill in lieu of compacted fill. The flowable fill has other common names, such as, unshrinkable fill, controlled density fill, flowable mortar, flowable fill, plastic soil-cement and soil-cement slurry (Du et al., 2006). This controlled low-strength filling material is made of cement, fly ash, water, sand, and typically an air-entraining admixture (NCHRP, 597). A significant requisite property of flowable fill is the self-leveling ability, which allows it to flow; no compaction is needed to fill voids and hard-to-reach zones (Abu-Hejleh et al., 2006). Therefore, the flowable fill is commonly used in the backfill applications, utility bedding, void fill and bridge approaches (Du et al., 2006).

The primary purpose of using flowable fill is as a backfill behind the abutment. CDOT has used the flowable fill backfill behind the abutment wall in an effort to reduce the approach

settlements since 1992 (Abu-Hejleh et al., 2006). The other new applications for the flowable fill are for use as a sub base under bridge approaches and a repair work of the approaches (Du et al., 2006). Historically, the application of using flowable fill as a sub base was first employed in Ohio by ODOT (Brewer, 1992).

In Iowa, the flowable fill is a favorable backfill used as a placement under the existing bridges, around or within box culverts or culvert pipes, and in open trenches (Smadi, 2001). Smadi (2001) also cited that the advantages of flowable mortar are not only due to its fluidity, but also due to its durability, requiring less frequent maintenance. Moreover, the flowable mortar is also easily excavated. Therefore, the maintenance works, if required, can be done effortlessly (Smadi, 2001). Figure 2.6 shows details of flowable mortar used under a roadway pavement.

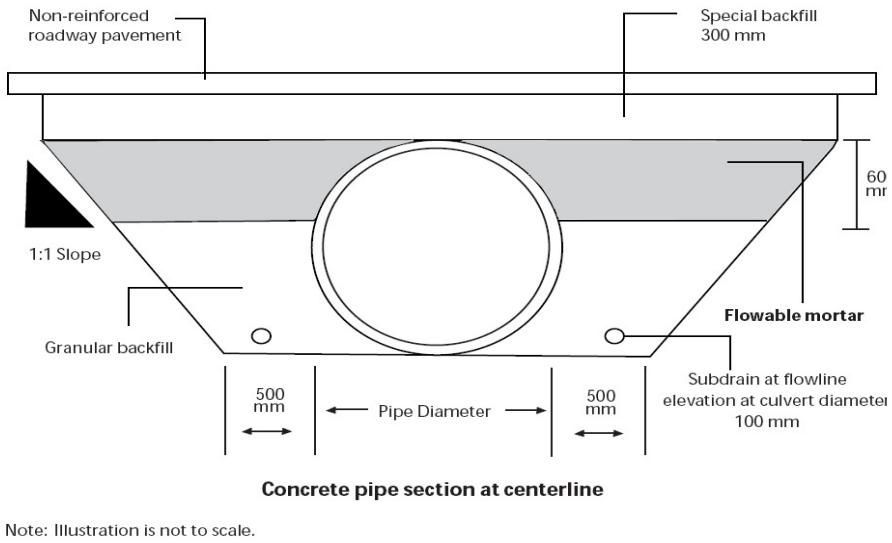


Figure 2.6 The flowable mortar used under a roadway pavement (Smadi, 2001)

In Texas, the flowable fill was used for the first time for repairing severe settlements of bridge approaches at the intersection of I-35 and O'Conner Drive in San Antonio in 2002 by TxDOT (NCHRP, 597; Du et al., 2006). For this practice, TxDOT used a specialized mixture

using flowable fill, which consisted of sand, flyash and water; no cement (Williammee, 2008). The compressive strength of cored samples indicated that the long-term strength and rigidity of the flowable fill were strong enough to serve this purpose (NCHRP, 597). After the mixture proportions were adjusted to have adequate flowability for the application, the flowable fill has shown a great success for repairing the approaches (Du et al., 2006 and Williammee, 2008). Recently, the flowable fill was used in the Fort Worth District in place of a flexible base beneath the approach slab. The 3 ft. deep flex base is prepared with Type 1 cement (2.4% by weight) as a base material as shown in Figure 2.7.

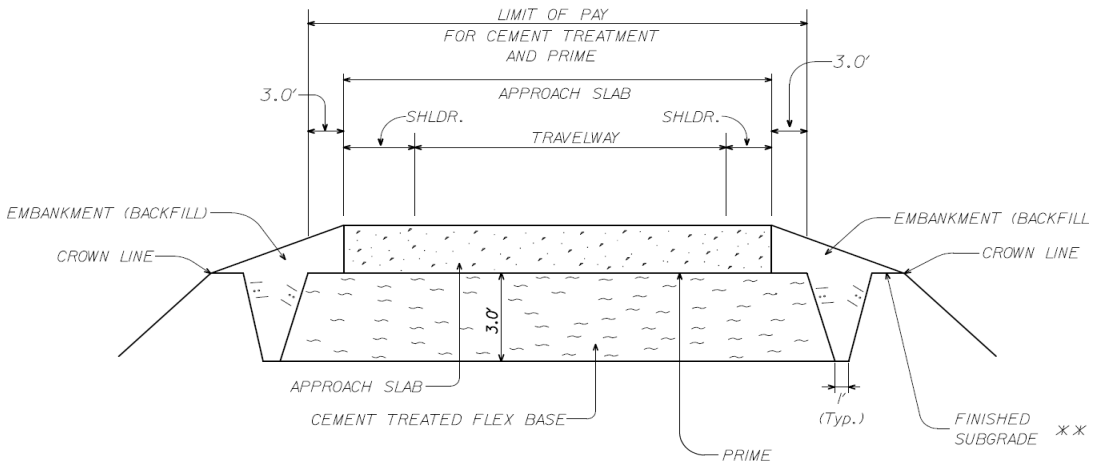


Figure 2.7 The flowable fill used as a base material (Du, 2008)

2.5.4 Other Methods

Several other techniques are also available to mitigate the settlement problem caused in the approach slab area and these techniques are discussed in the following.

2.5.4.1 Precambering

If the approach pavement settlement cannot be controlled economically, a pre-cambered roadway approach may be applied (Tadros and Benak, 1989). Hoppe (1999)

recommended implementing pre-cambering of bridge approaches for up to a 1/125 longitudinal gradient. The pre-cambering is used to accommodate the differential settlement that will inevitably occur between a structure constructed on deep foundations and adjoining earthworks.

Briaud et al. (1997) recommended pre-cambering with gradient values of less than 1/200 of the approach slab length to compensate for the anticipated post-construction settlements. The pre-cambered design utilizes a paving notch that supports a concrete slab. The notch must be effectively hinged, which allows the concrete slab moving radially. The flexible pavement over the slab will absorb some movement below it but not to a great extent (Briaud et al, 1997). The pre-cambered approach system also requires an accurate assessment of settlement potential (if possible). The pre-cambered approach design could be specified in situations where time is not available for more conventional settlement remediation, such as preloading, wick drains, and others (Luna, 2004).

Wong and Small (1994) conducted laboratory tests to investigate the effects of constructing approach slabs with an angle from the horizontal on reducing the bump at the end of the bridge. It was found that horizontal slabs suffered a rapid change in surface deformation with the formation of obvious bumps, while pre-cambering the slabs with angles of 5° to 10° provided a smoother transition.

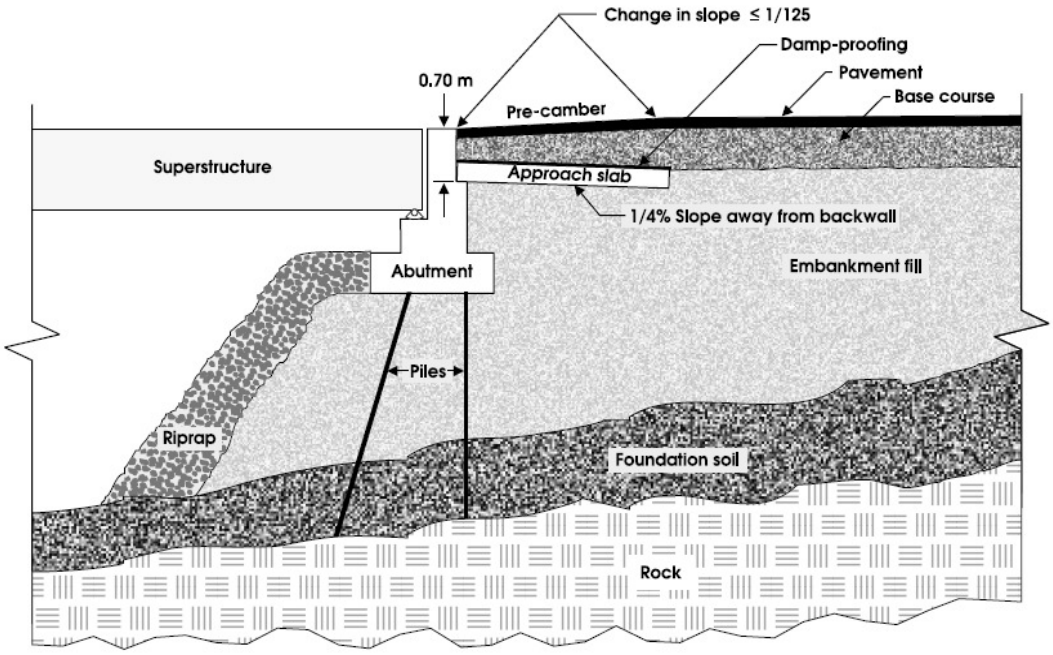


Figure 2.8 Pre-cambered Approach Design (Hoppe, 1999)

2.5.4.2 Lightweight Fill Materials

The lightweight materials such as Expanded Polystyrene (EPS) Geofoam and Expanded Clay Shale (ECS) can be used either as a construction embankment fill material for new bridge approach embankments or can be used as a fill material during the repair of distressed approach slabs.

2.5.4.3 Expanded Polystyrene (EPS) Geofoam

Expanded Polystyrene (EPS) Geofoam is a lightweight material made of rigid foam plastic that has been used as fill material around the world for more than 30 years. This material is approximately 100 times lighter than conventional soils and at least 20 to 30 times lighter than any other lightweight fill alternatives. The added advantages of EPS Geofoam including reduced loads on underlying subgrade, increased construction speed, and reduced lateral stresses on

retaining structures has increased the adoptability of this material to many highway construction projects. More than 20 State DOTs including Minnesota, New York, Massachusetts, and Utah adopted the EPS Geofoam to mitigate the differential settlement at the bridge abutments, slope stability, alternate construction on fill for approach embankments and reported high success in terms of ease and speed in construction, and reduced total project costs.

Lightweight EPS Geofoam was used as an alternate fill material at Kaneohe Interchange in Oahu, Hawaii while encountering a 6m thick layer of very soft organic soil during construction. 17,000 m3 of EPS Geofoam was used to support a 21 m high embankment construction (Mimura and Kimura, 1995). They reported the efficiency of the material in reducing the pre- and post-construction settlements. Figure 2.9 shows the construction of the embankment with the EPS Geofoam.

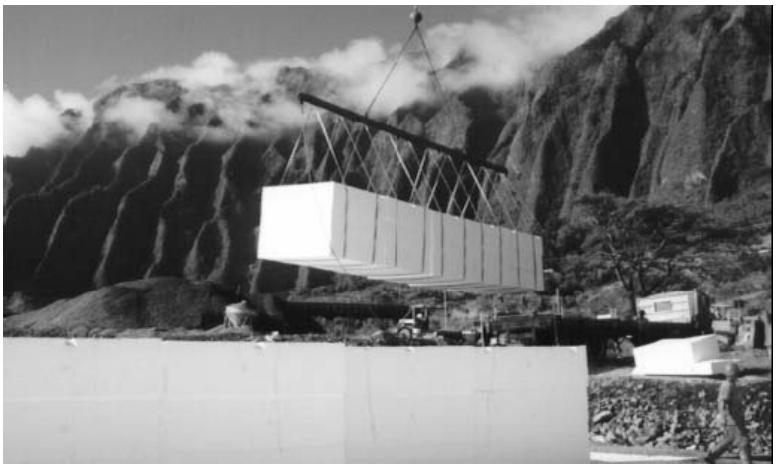


Figure 2.9 Emergency Ramp and High Embankment constructed using EPS Geofoam at Kaneohe interchange in Oahu, Hawaii

2.6 Summary

This chapter presented a thorough review of the literature review on settlement at the bridge approach. The definition of the settlement at the bridge approach problem is firstly presented following with the magnitude of the bump tolerance. Afterward, major mechanisms

causing the bump problem are introduced. The primary sources of the problem broadly divided into four categories are material properties of foundation and embankment, design criteria for bridge foundation, abutment and deck, construction supervision of the structures, and maintenance criteria. (Hopkins, 1969, 1985; Stewart, 1985; Greimann et al., 1987; Laguros et al., 1990; Kramer and Sajer, 1991; Ha et al., 2002; Jayawikrama et al., 2005; White et al., 2005, 2007).

Major focus of this chapter was given to the techniques used to mitigate the bump problem, which was presented in two sections; the techniques for new bridge construction, and the measures for distressed approach slabs. The techniques for new bridge construction can be divided into various groups such as improvement of foundation soil, improvement of backfill material, design of bridge foundation, design of approach slab, and effective drainage and erosion control methods. The main focus was on the techniques to improve the embankment backfill material.

The maintenance measures for distressed approach slabs are normally used as remedial measures after problems are detected. The most important techniques fallen into this category are pressure grouting under the slab, slab-jacking or mud-jacking technique, the Urethane method, and compaction or high pressure grouting. It can be noted that many of these measures could be also applied for improvement of backfill material in new bridge constructions.

CHAPTER 3

MAINTENANCE MEASURES FOR DISTRESSED APPROACH SLABS

3.1 Introduction

The bridge approach maintenance costs in the United States are estimated to be at least $100 million per year (Briaud et al., 1997; Nassif et al., 2002). The best practice to minimize the occurrence of bridge bumps is to establish up-to-date maintenance activities, by scheduling periodic repair activities in addition to the required occasional maintenance (Dupont and Allen, 2002). Hence it is important to understand the different mitigation techniques in use to repair the distressed bridge approach slabs. This chapter presents an overview of the repair methods adopted for treatment of settled bridge approach slabs in various districts in Texas. Also, the results and details of the geophysical studies performed to implement a few of these technologies are summarized.

3.2 Bridge Identification and Repair

A total of four bridges with distressed approach slabs in Texas were identified. Table 3.1 gives the identification and location of the bridges along with the mitigation technique either adapted or proposed as a part of the present research.

Table 3.1 Distressed bridges and mitigation techniques adopted

S. No.	Identification	Location	Mitigation Technique Proposed
1	FM 1947	Hillsboro, Waco Dt.	Urethane Injection
2	US 67/SH 174	Cleburne, Fort Worth Dt.	Soil Nailing
3*	SH 6	Quanah, Childress Dt.	Geo Foams and Flowable Fills
4*	IH 410	San Antonio, San Antonio Dt.	Flowable fills

* Repair solution recommended but not implemented

Each subsection in the Chapter presents the repair solutions and geophysical studies performed.

3.2.1 FM 1947 Hillsboro Waco Dt. Texas

Bridge approach slab repair work along a bridge crossing Aquilla Lake on FM 1947, Hillsboro, Waco district, Texas, was the first one studied as a part of this research. The location of the bridge is shown in Fig 3.1. At this bridge site, polyurethane injection technique was used to uplift the settled bridge approach slab.

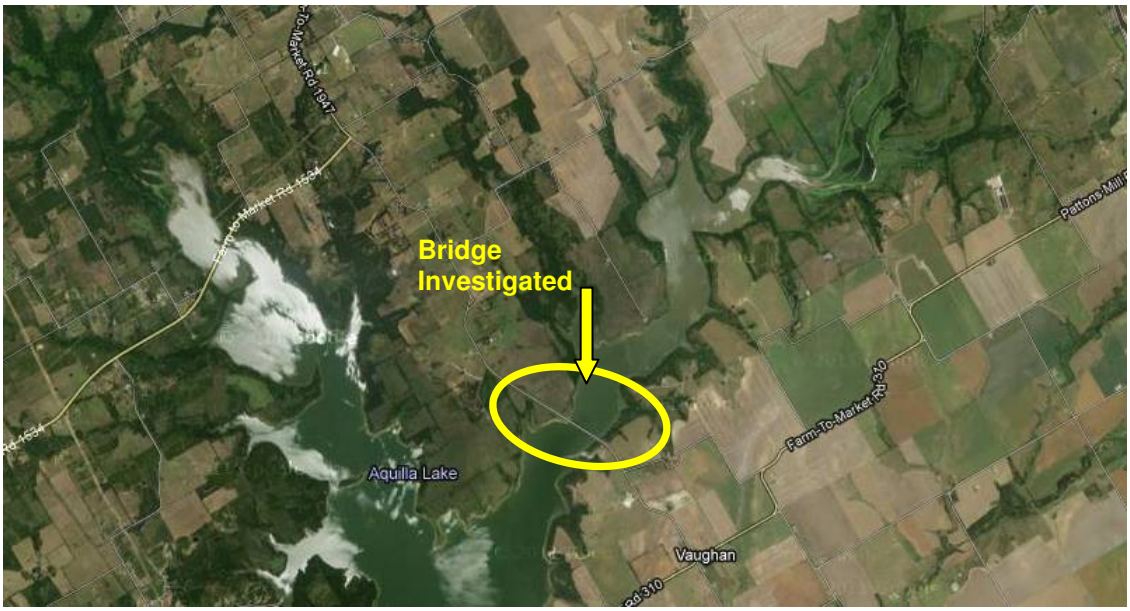


Figure 3.1 Bridge site location, FM 1947 Hillsboro

Urethane is a high density polymer resin which can be injected through tiny holes (usually 5/8 in. diameter) made through pavement slabs into void spaces present beneath them, adjacent to bridge abutments etc. Once injected, Urethane will expand, forming foam like structure which covers more than 90% of void space and hardens in a very short period of time (usually 15 min. or less). This technique is usually performed in a square or triangular pattern of

holes to fill the void spaces, re-aligning the pavement structures into their original position. Figure 3.2 presents a sketch of the pavement and bridge sections observed in this study.

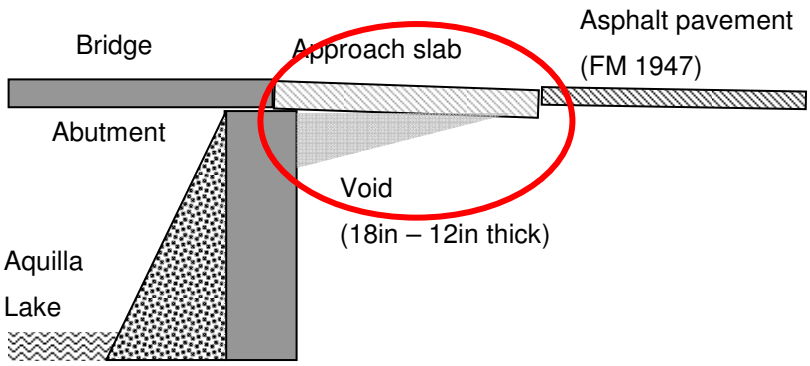


Figure 3.2 Problem definition sketch of FM 1947 Bridge site, Hillsboro, Texas

The urethane injection work at this site has been carried out by URETEK Company, USA. Before the actual injection process has begun, elevation levels were taken on firm bridge portion and settled approach slab. A leveling thread was run through the settled approach slab and firm adjacent pavement to estimate the required amount of uplift of the approach slab. The total settlement of the approach slab is around 4 in. before the repair. The depth of void adjacent to the bridge abutment varied from 2 in. to 18 in., gradually increasing towards the abutment. Figure 3.3 and Figure 3.4 show the settlement and void formation respectively. Figure 3.5 gives a closer view of the void under the slab.

Figure 3.3 Settlement of the approach slab.

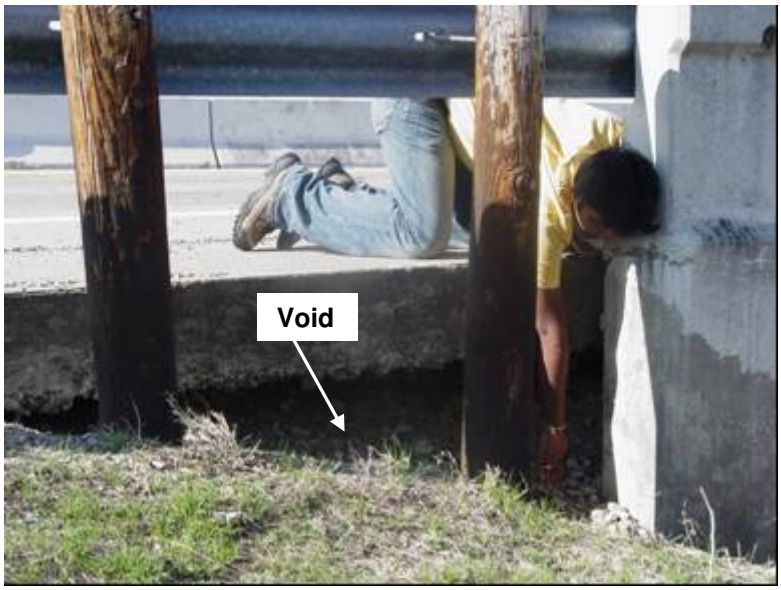


Figure 3.4 Void devoloped adjacent to abutment

Figure 3.5 Closer view of the void gap

The actual injection process began with drilling tiny holes of size 5/8” diameter into the approach slab in a rectangular pattern to inject the polymer/Urethane. Urethane polymer expanded forming foam and hardened in place in less than 15 min. and lifted the settled approach slabs to the required elevation. Since this complex material is very light weight, the pressure coming to the subsoil due to the expanded foam is expected to be very low. Besides, the time required for this treatment process was very short (total process time is in hours) when compared to the other conventional treatment techniques for this kind of situation such as cement grouting, asphalt overlay or complete excavation and replacement of existing approach slabs. Figures 3.6 to 3.12 show the process of Urethane injection beginning from the drilling of holes to the level up of sections after the uplift of the settled approach slab.

Figure 3.6 Drilling holes into the approach slab

Figure 3.7 Another view of drilling holes

Figure 3.8 Sealing the void from exterior face to protect loss of injected Urethane

Figure 3.9 Taking levels during the injection process to check for extra injection

Figure 3.10 Uplift of approach slab during the injection process

Figure 3.11 Another view of uplift of approach slab

Figure 3.12 Level-up of uneven sections with cold asphalt mix

Though there is no technical data readily available on this technique such as quality assessment and quality control studies, the post performance of the approach slabs is found to be good. There are a few bridges that were repaired in Houston District, Texas several years back (more than ten years) and the performance of these bridges appear to be satisfactory. One important item of interest when using urethane injection is to determine the amount of urethane quantity that needs to be injected as this estimation is important in the cost analysis of the repair strategies. Considering this, TXDOT amended its special specification # 3043-001 which specifies the special provision for “Raising and Under Sealing Concrete Slabs” as the quantity of polymer resin utilized will be calculated by one of the following methods:

Payment will be made according to the actual quantity of polymer resin used in the work by determining the weight of material placed by measuring the depth of polymer resin in the holding tanks before and after each day’s work. A professional engineer and a site engineer

must approve the calculation method which is based on certified tank measurements, unit weight of each component of resins used in the work.

The repaired bridge in Waco district was monitored for three years and the settlement of the slab was always within in allowable limits (less than ¼ in.). However, there was a recent overlay on the approach slab along the FM 1947 in June 2011; since the overlay was performed all along the pavement section for several miles, it could be mentioned that the repaired approach has performed well without any additional distress.

3.2.2 US 67/SH 174 Cleburne Fort Worth Dt. Texas

Repair work for the distressed bridge approach slabs of the both ends of bridge along US 67, crossing over SH 174 in Cleburne, Fort-Worth Dt. Texas was performed recently. The identified bridge has slabs placed on embankments whose heights are around 35 ft. The location of the bridge and the bridge photo are given in Figure 3.13 and Figure 3.14 respectively.

Figure 3.13 Bridge site location US 67/SH 174 Cleburne, Texas

Figure 3.14 Bridge along US 67, crossing over SH 174 in Cleburne

The bridge was reported to be the first one using the MSE wall abutment walls with geogrids as reinforcing elements. Over the years, the approach slabs settled considerably. Several mitigation methods were attempted to mitigate this settlement and these methods include Micropiles and Overlays and these were not successful. The local area office informed the research members that the approach slab is overlaid several times, however settlements did not subside.

As a part of the research, laboratory tests were performed on the fill material under the approach slab. This material is classified as non plastic, poorly graded sand. The problems for the distress in the approach slab are probably due to the erosion of fill under the slab and the settlement of the underlying foundation. The failure of the MSE wall and the erosion of soil under the slab are shown in Figures 3.15 and 3.16.

Figure 3.15 Failure of the MSE wall

Figure 3.16 Erosion of soil under the slab

In order to address the movements of MSE wall and enhances the stability of the MSE walls, soil nailing technique was recently adopted by the area office. Soil nails are slender reinforcement elements, generally rebars, inserted into the soil at predetermined locations based on the design to provide shoring and stability. Holes are drilled for placing the

reinforcement and once placed filled with grout. The rebars are placed at a slightly downward inclination to the ground surface and are placed at regularly spaced intervals. Once the rebars are placed, a wire mesh is placed on the soil face and is shotcreted from the exterior.

The Soil nailing technique was carried out by Olden Inc. at the bridge site. Reinforcement bars confining to ASTM A615, grade 60 (#6 rebars) with a head plate confining to ASTM A36 of dimensions 5”x5”x⅜” were used as soil nails. Coinc 528NS grout with a compressive strength of 3000 psi after 7 day curing period was used for grouting. The drilling of the holes for placing the soil nails is shown in Figure 3.17.

Figure 3.17 Drilling of holes to place the soil nails

The soil nails were placed at a horizontal spacing of 6 ft. and a vertical spacing of 4 ft. interval. The length of each nail is 16 ft. After the placement of the nails, the hole was completely grouted. Samples of the grout were collected to test the compressive strength of the grout. The placing of the soil nails and the grouting the drilled hole is shown in Figure 3.18.

Figure 3.18 Placing of nails and grouting the holes

A welded wire fabric reinforcement confining to ASTM A185 with dimensions 4” x 4” (W1.4 x W1.4 10GA) was placed along the soil face and is shotcreted with a shotcrete strength of 4 ksi at 28 days after curing. The shotcreted exterior is shown in Figure 3.19. The total soil nailing process on both ends of the bridge was completed in ten days.

Figure 3.19 Shotcreting the exterior

Since the soil nailing technique was recently adopted in the early summer of 2011, further monitoring of the bridge is needed to address the use of this technique to control

approach slab settlement that is induced by the MSE wall fill erosion and potential settlement of foundation subgrade.

In the following sections, two comprehensive geophysical and geotechnical explorations were performed to understand the causes of settlements of slabs at two locations. A few treatment methods were suggested; however they are not implemented due to the needs for permanent solutions at this site. Results of investigations are presented in the following.

3.2.3 SH 6, Quanah, Childress District Texas

A series of field tests were carried out by the UTEP research team to assess the condition of a bridge approach on SH 6 road near Quanah, Childress Dt., Texas. A ground penetrating radar system (GPR) was used to evaluate the bridge approaches and to detect potential voids or anomalies underneath them. Field tests were conducted on the northbound and southbound lanes. Figure 3.20 gives the location of the bridge investigated.

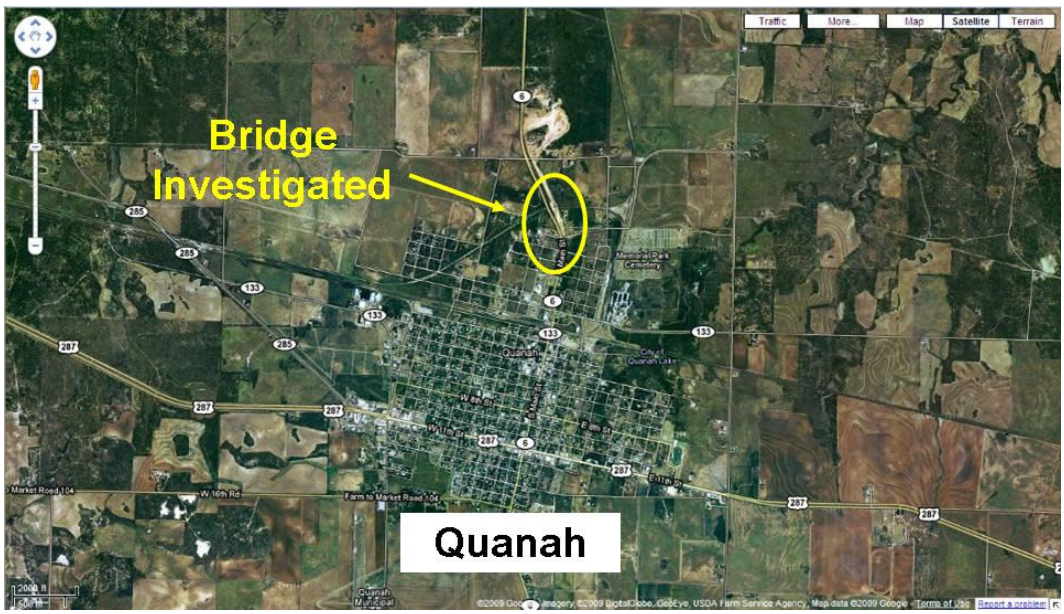


Figure 3.20 Bridge Site Location, SH 6 Quanah

Ground Penetrating Radar (GPR) is a geophysical nondestructive technique that uses electromagnetic pulses to test, characterize, or detect subsurface materials based on changes in electrical and magnetic properties of the subsurface layers. GPR works using short electromagnetic pulses radiated by an antenna which transmits these pulses and receives reflected returns from the pavement layers. The reflected pulses are received by the antenna and recorded as a waveform. As the equipment travels along the pavement, it generates a sequence of waveforms or linescans. These waveforms are digitized and interpreted by computing the amplitude and arrival times from each main reflection. The reflections of these waves at interfaces and objects within the material are analyzed to determine the location or depth of these interfaces and buried objects, and to determine the properties of material. Whenever applicable, GPR can be employed as a rapid nondestructive tool for evaluation of geometrical and material properties of structural components.

The test setup used in this study is shown in Figure 3.21. The equipment used for the GPR survey consists of a GSSI SIR-20 (SIRveyor) two-channel data acquisition unit controlled by a laptop computer, a ground-coupled antenna and a survey wheel attached to the antenna. For this study, three different antennas were used: a 1.6 GHz (GSSI Model 5100), a 400 MHz (GSSI Model 5103) and a 200 MHz (GSSI Model 5106). The three antennas were used to cover different penetration depths, identify precisely pavement interfaces and also detect presence of utilities or anomalies at different depths. The maximum depth of penetration of each antenna reaches to about 18 inches, 4 ft and 8 ft respectively, depending on the conductivity of the materials underneath.

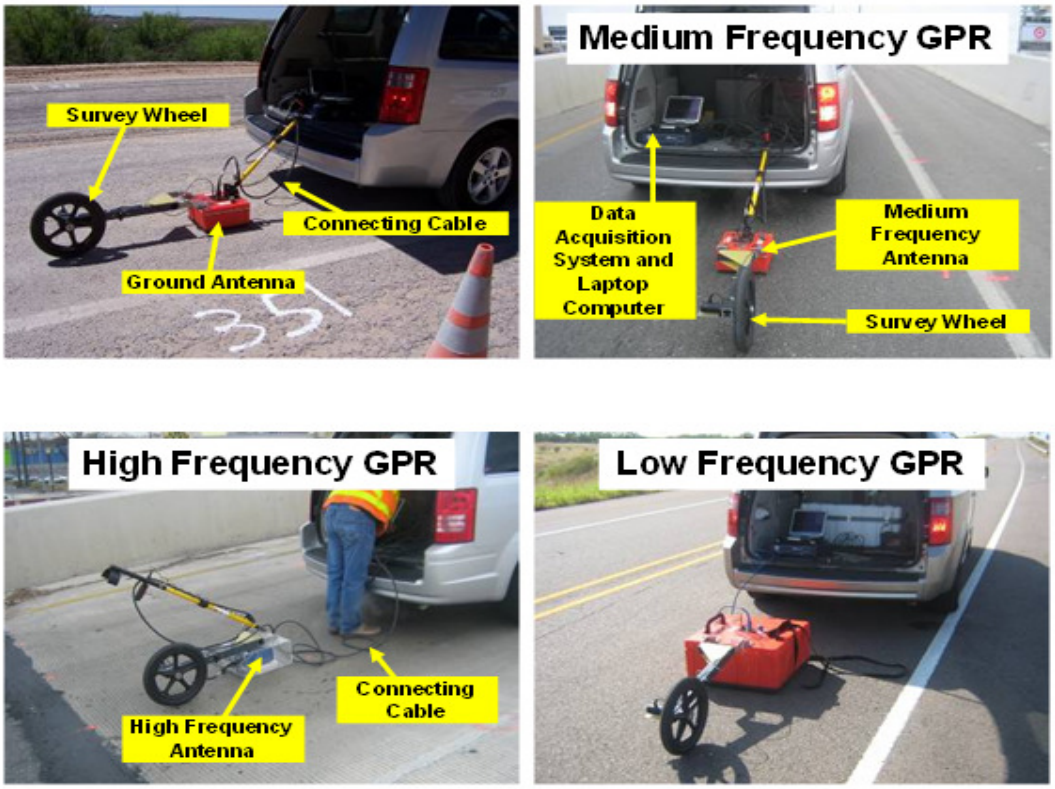


Figure 3.21 GSSI GPR System and Antennas used in this study

The schematic of the test section along SH6 in Quanah is shown in Figure 3.22 and the views from both ends of the section and north and south approach slabs are illustrated in Figure 3.23. The shoulders, main lanes and selected concrete slabs from the bridge abutments were investigated. Linescans with the low (200 MHz) and high (1.6 GHz) frequency antennas were obtained along the center of the shoulder and main lanes in both directions. The lengths of the survey lines along the north end of the bridge were 1600ft., while the survey lines on the south end were 800ft. long. In addition, four linescans with the high frequency antenna were obtained on each of the four abutments (named from 1 to 4) to evaluate the presence and extent of anomalies and voids on the selected slabs. Linescans of two of the four lanes were obtained on

the outer edges of the abutment and the other two were equally distributed on the radial direction of each abutment.

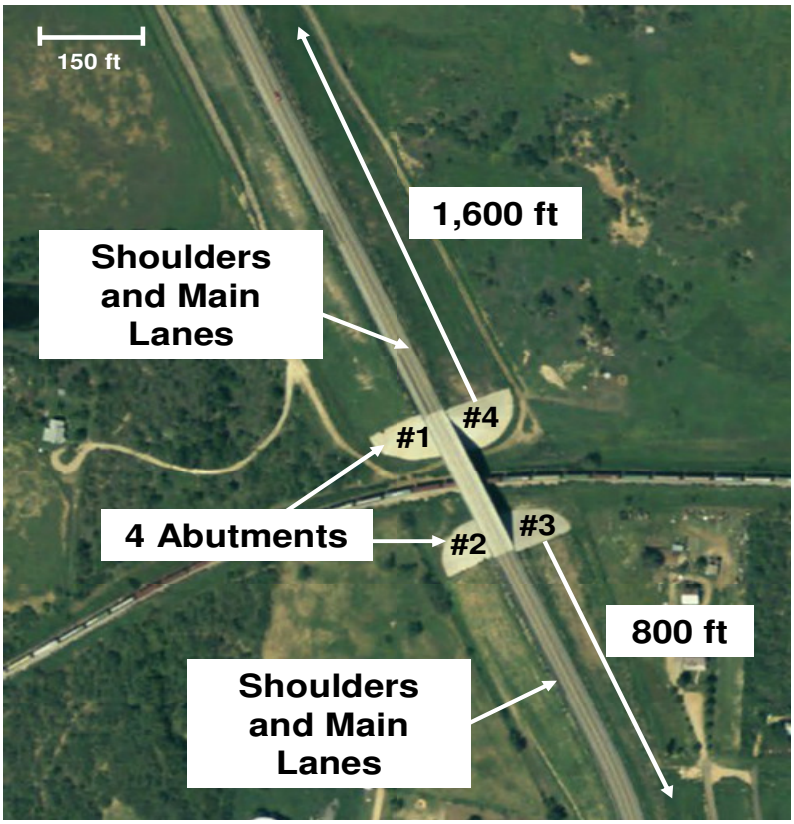
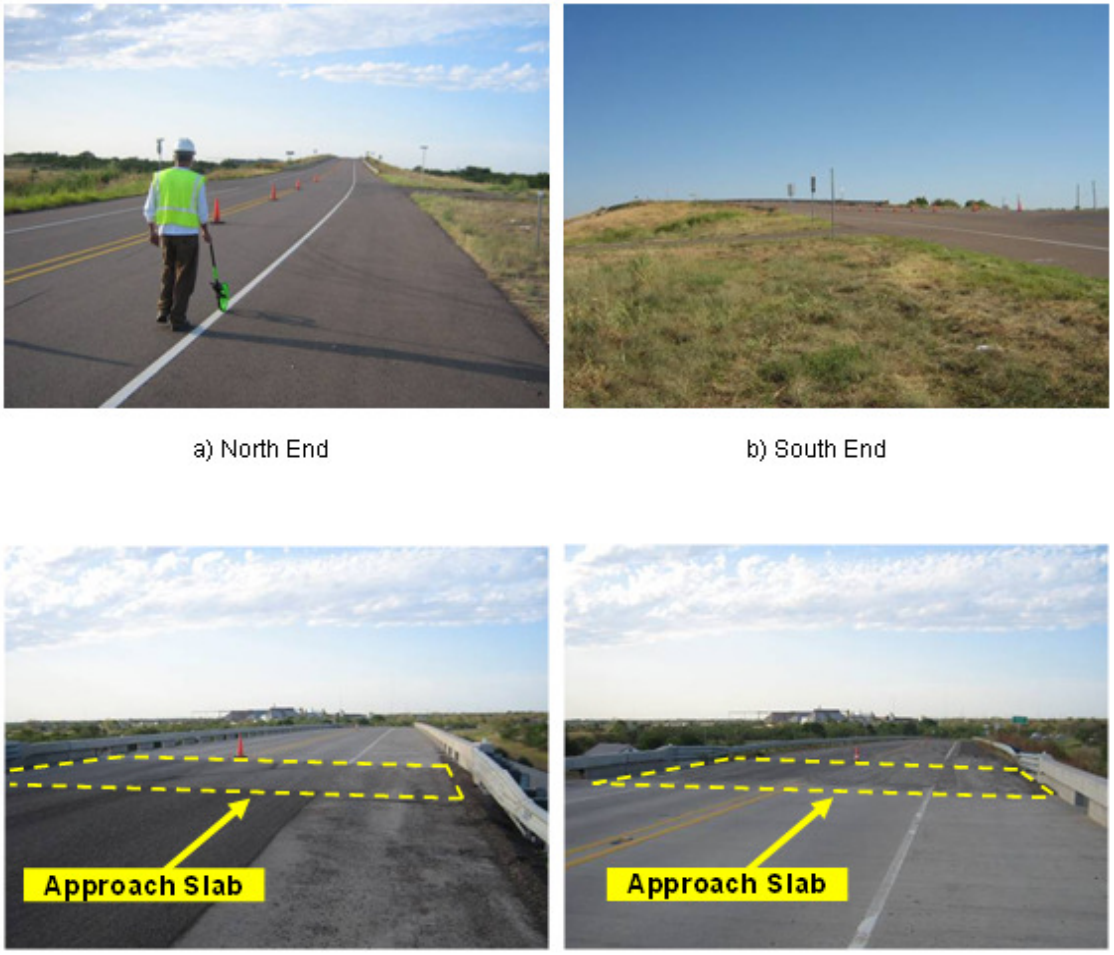


Figure 3.22 Test section along SH 6 in Quanah



a) North End

b) South End

c) North Approach Slab

b) South Approach Slab

Figure 3.23 Field Pictures from Quanah Bridge

A typical linescan for the test conducted on the southbound lanes obtained with the high frequency GPR antenna is shown in Figure 3.24. The horizontal axis relates to distance measured from the start point (ft.) and the vertical axis indicates estimated depth using a typical dielectric constant of 6.5. This linescan shows the last 20 ft. of the approach slab (total of 30 ft. long) and the pavement structure afterwards. The pavement structure on the approach slab consisted of a hot mix asphalt (HMA) layer on top of a concrete slab of about 10 in. thick. The

approach slab, HMA-base interface, and subgrade-base interface are identified in the figure. For the approach slab, the bottom of the HMA and the transverse reinforcement within the slab are marked. Based on the linescan, the thickness of the HMA was approximately 2.5 in and the reinforcement appeared about 5 in. below that.

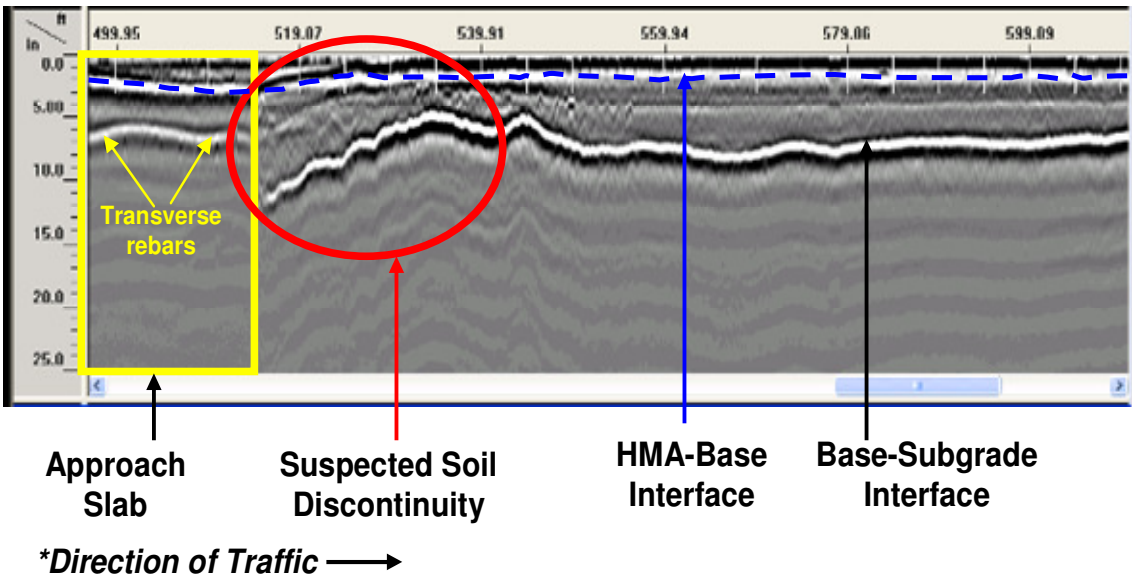


Figure 3.24 Typical Linescans from Quanah Bridge with High Frequency Antenna

In addition, an area of discontinuity immediately after the approach slab is highlighted. This area was approximately 40 ft long and showed erratic base-subgrade interface depth that ranged from 5 to 13 inches. For the rest of the linescan this interface appears at about 8 in. The HMA-base interface appears to be at a depth of approximately 2.5 in. Under these layers, several other horizontal interfaces are dimly observed. These other layers can be assumed as different strata within the subgrade.

Similar to the high frequency antenna, a typical linescan obtained with the low frequency antenna along the same area is shown in Figure 3.25. In this case, the horizontal scale is different because the start point was located a few feet before the approach slab.

Strong reflections are observed at the beginning of the linescan that correspond to the bridge slab. Due to the lower frequency of the antenna, details of the approach slab are not as clearly identified as with the other antenna. The suspected soil discontinuity is also present that is extended deeper than the previous linescan. Like the high frequency antenna, several other horizontal interfaces are observed, but with stronger reflections. Similarly, these layers can be assumed as different strata within the subgrade.

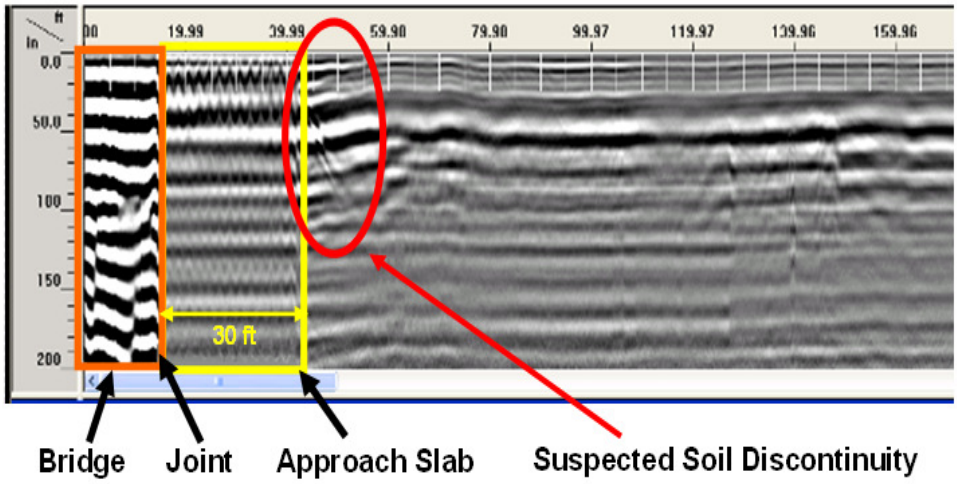


Figure 3.25 Typical Linescans from Quanah Bridge with Low Frequency Antenna

A sample linescan along the second slab of Abutment #3 is shown in Figure 3.26. Some anomalies along the last 10 ft (closer to the top of the abutment) were observed. These may reveal the presence of voids or areas of concerns.

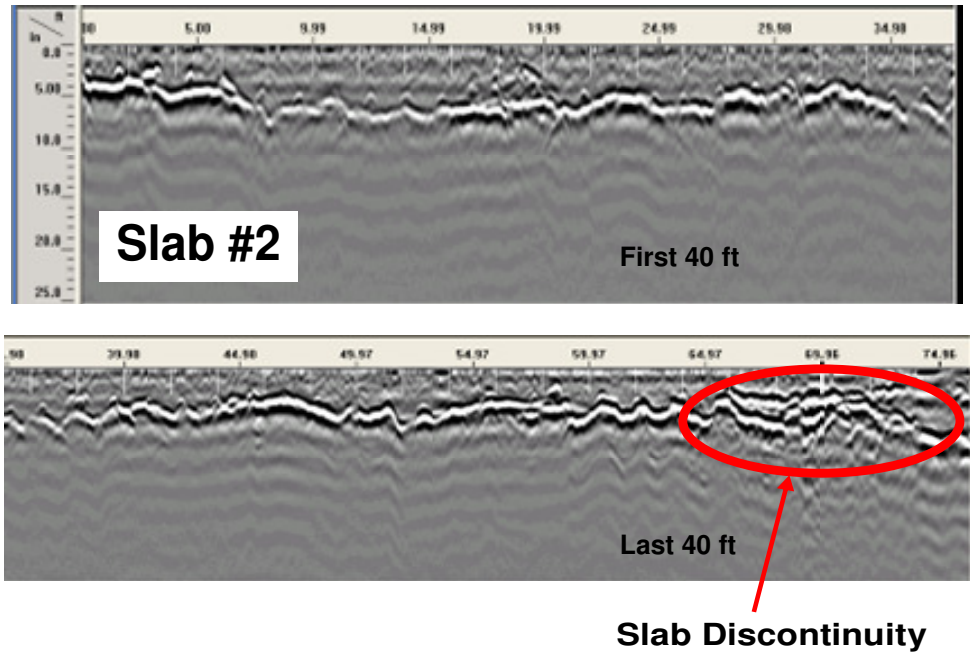


Figure 3.26 Typical Linescans from Quanah Abutment

The overall results from this survey are summarized in Figures 3.27 and 3.28 for the North and South sides of the bridge, respectively. GPR scans on most critical areas are also shown. The locations of possible voided areas on concrete abutments are also depicted. Based on the GPR data, areas of concern along the north side are located mainly on the southbound shoulder and lane and also about 30 ft from the approach slab. Only one area on the south side contiguous to the approach slab seems to show some anomalies. For the case of the concrete abutments, the outer lines of Abutments #1 and #2 appear to have the longest voided areas.

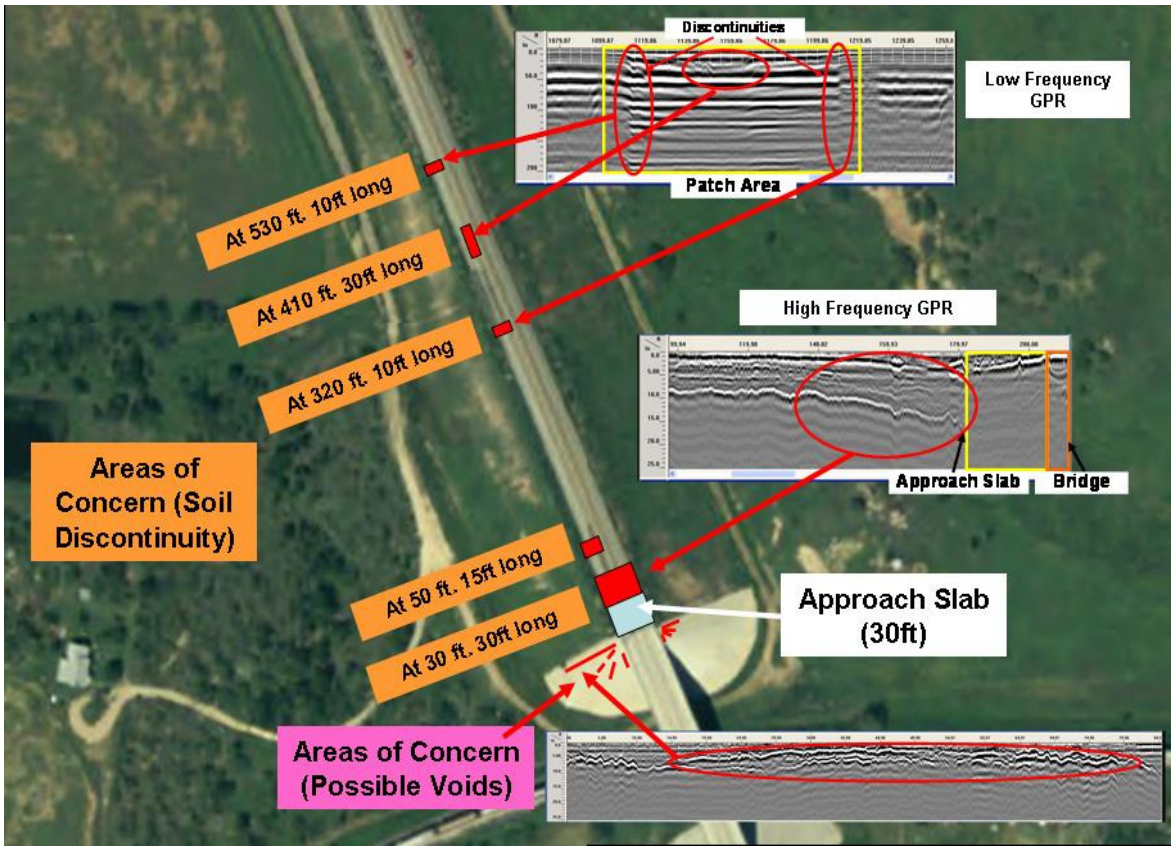


Figure 3.27 Overall results from Quanah Bridge (North Side)

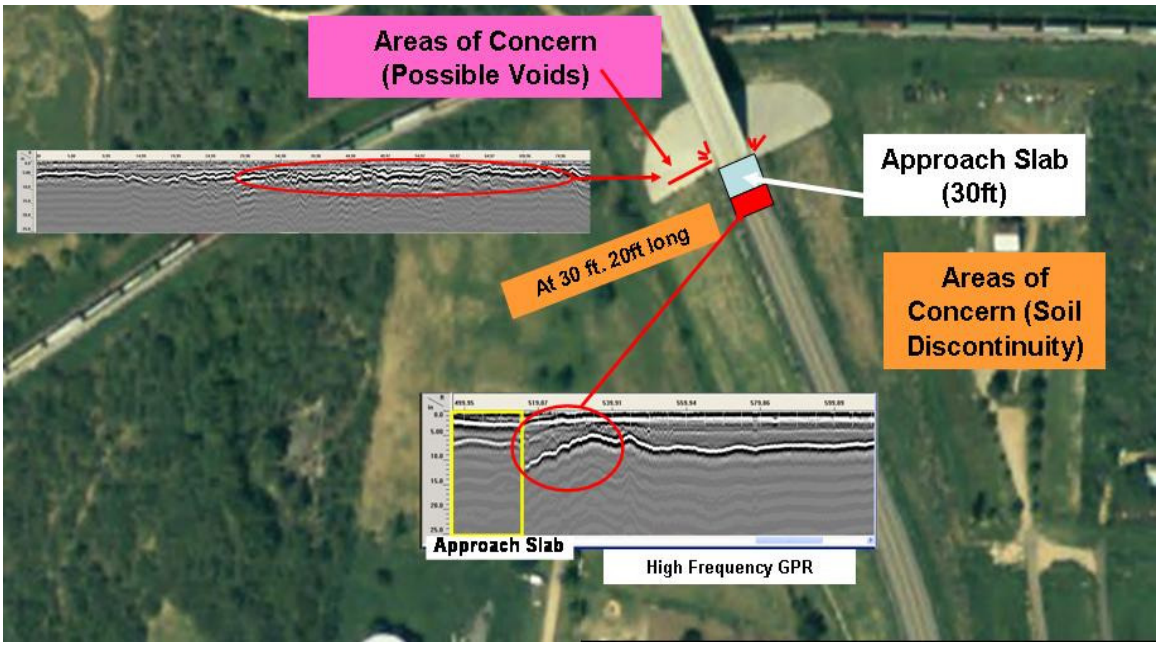


Figure 3.28 Overall results from Quanah Bridge (South Side)

Potential Void Volume Estimation under Approach Slabs

An estimation of the suspected voids under the approach slabs, immediately before the bridge deck was carried out. For this estimation, detailed GPR scans on the approach slabs were evaluated. As an example, Figure 3.29 shows a linescan of the southbound shoulder. The HMA-concrete interface, transverse rebars, and the bottom of the approach slab are identified at several selected points. With the assumed dielectric constant of the pavement structure, estimated depths were obtained and are presented on Figure 3.30. The suspected void under the approach slab adjacent to the bridge joint is also depicted. This suspected voided area was measured by calculating the compound average difference of the bottom of the approach slab away from the bridge joint, and the measured from the linescan adjacent to the bridge joint.

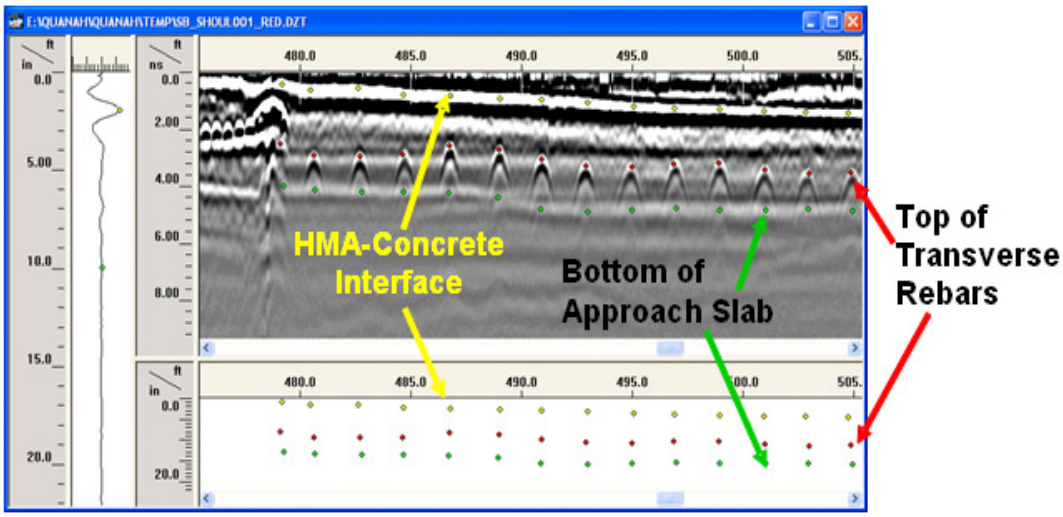


Figure 3.29 Selection of points from GPR Linescan

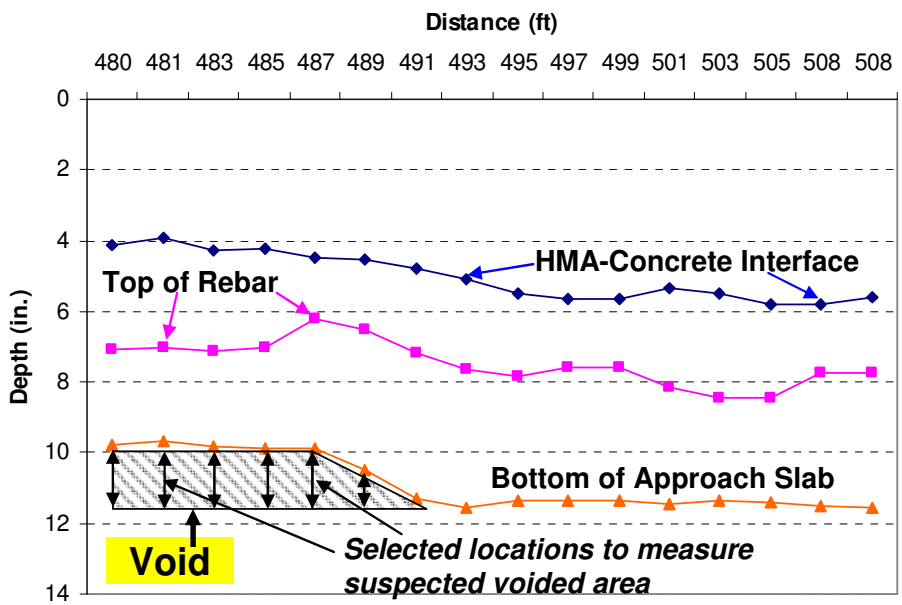


Figure 3.30 Estimated Depth Profile

In the above case shown in Figure 3.30, the voided area extends about 11 ft from the bridge joint with a trapezoidal shape. The estimated volume was obtained by multiplying this area by the width of the shoulder (8 ft in this case). The same process was implemented on the rest of the shoulders and main lanes and final void volumes were obtained and summarized in Table 3.2. A width of 12 ft was used for the case of the main lanes. From Table 3.2, the region under the south bound lane seems to have larger voided areas.

Table 3.2 Voids under Approach Slabs

	Estimated Void Volume (ft3)	
	Before Bridge	After Bridge
SHOULDER SB	3.7	7.8
LANE SB	23.5	15.4
SHOULDER NB	10.4	6.5
LANE NB	13.9	15.5

The same study was conducted for the four concrete slabs investigated on each abutment. Estimated void volumes are presented on Table 3.3. A width of 4 ft was assumed for each slab to obtain the volume. Abutments #1 and #4 exhibit larger void volume under the concrete slabs.

Table 3.3 Voids under Abutment Slabs

	Estimated Void Volume (ft3)				
	Slab 1	Slab 2	Slab 3	Slab 4	
Abutment 1	1.9	3.3	8.6	10.8	*North Side SB Lane*
Abutment 2	1.2	1.9	3.0	5.4	*South Side SB Lane*
Abutment 3	0.0	8.4	3.0	3.5	*South Side NB Lane*
Abutment 4	8.0	3.8	2.4	5.9	*Noth Side NB Lane*

Based on the estimated void volume, use of Geofoam blocks covered with a Geomembrane to protect from leaching oils on the north side of the bridge and use of Flowable fill on the south side of the bridge is proposed. The proposed mitigation techniques were not implemented as the long term performance of these techniques is still in the research phase and the local district is exploring for permanent solutions.

3.2.4 IH 410 San Antonio San Antonio Dt. Texas

A series of field tests, similar to the tests conducted on SH 6 in Quanah, using a Ground Penetration Radar were carried out to assess the condition of a bridge approach on IH 410 in San Antonio. In addition cores were collected from the test locations and Dynamic Cone Penetration (DCP) test was conducted at the core locations. The results of the GPR system and DCP test are presented and the volume of voids under the approach slabs is estimated. The location of the identified bridge is given in Figure 3.31.

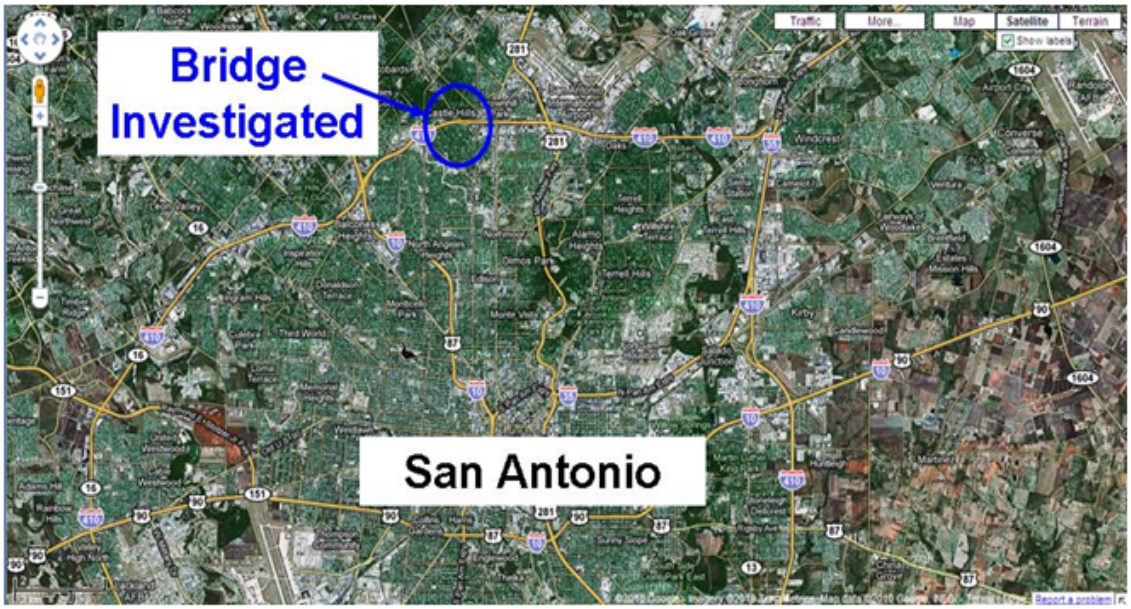


Figure 3.31 Bridge site location, IH 410, San Antonio

The overall view of the section is shown in Figure 3.32 and the views of the investigated west and east approach slabs are illustrated in Figure 3.33. Both slabs were 20 ft in length. Five longitudinal and five transverse linescans were obtained per each approach slab. The location of each line is detailed in Figure 3.34. GPR surveys were performed with a medium (400 MHz) and high (1.6 GHz) frequency antennas. The longitudinal surveys were carried out on the approach slab and also on 50 ft of the bridge ramp.

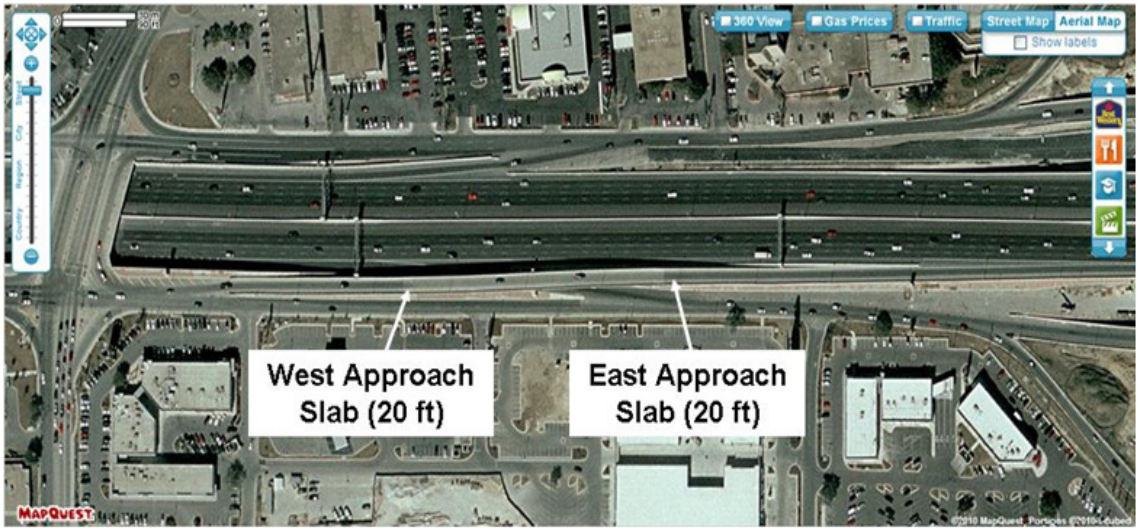


Figure 3.32 Overall section view of San Antonio site

Figure 3.33 Sectional views of west and east approach slabs

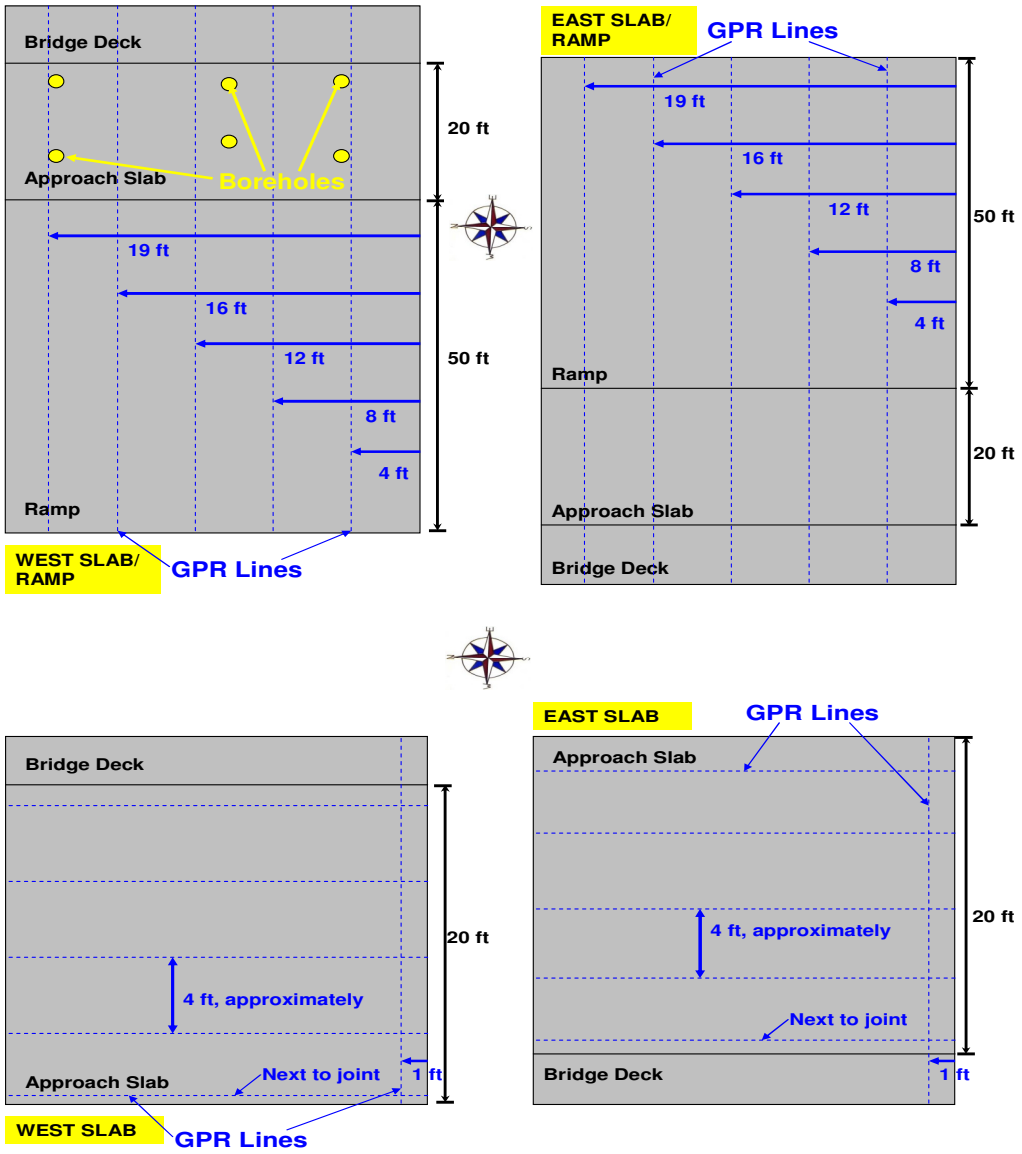


Figure 3.34 Detailed location of each line

A typical linescan obtained with the high frequency GPR antenna is shown in Figure 3.35, for the test conducted on the west approach slab. The Bridge, approach slab and ramp were clearly identified on the linescan. For the ramp, several layers at depths of about 2.5, 5, 8 and 12 in. from the top can be observed. A stronger reflection is also detected at about 18 in.

The first four reflections were attributed to different HMA interfaces that might represent several asphalt layers. What appears to be the HMA-embankment interface turns up at a depth of about 18 in. Some anomalies adjacent to the approach slab were also observed. For the approach slab, the transverse reinforcement is clearly detected at two different depths: between 4 and 6 in. and at 14 in. approximately. The bottom of the pavement structure was detected at about 18 in.

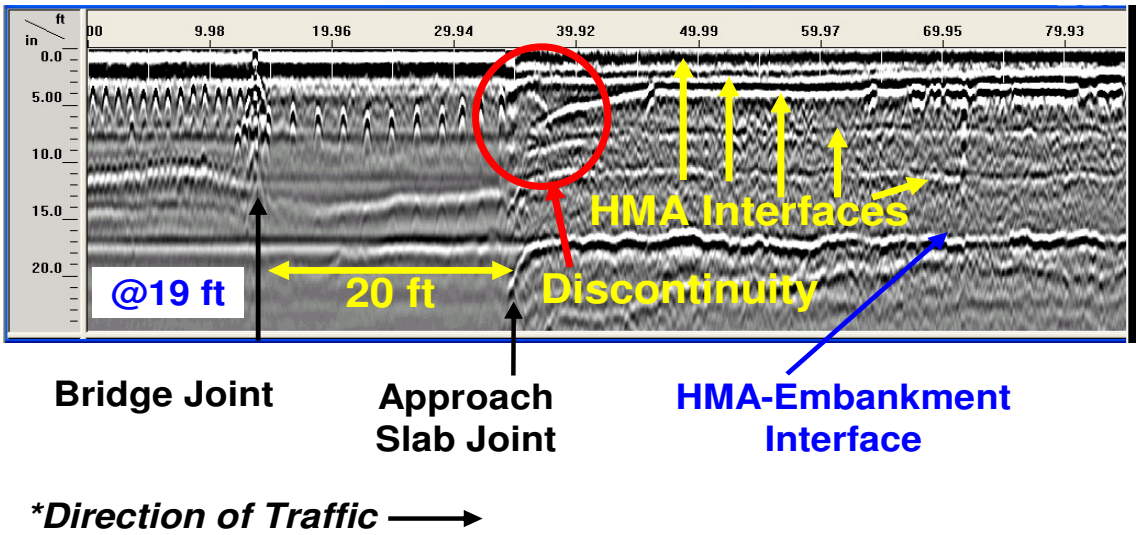


Figure 3.35 Typical linescan of west approach slab with High Frequency Antenna

GPR results from the medium frequency antenna are presented in Figure 3.36. Similar to the high frequency antenna, bridge, approach slab and ramp are clearly identified. In this case, HMA interfaces are not as visible as with the high frequency antenna, but the HMA-embankment interface is easy to distinguish. Under this layer, several other horizontal interfaces were observed. These other layers can be assumed as different strata within the subgrade. On the approach slab, the first level of transverse reinforcement is easy to identify, but not for the deeper level. In addition, an anomaly in the subgrade material immediately after

the approach slab is highlighted. This area was approximately 30 ft long and showed erratic interfaces at depths of 75 in. approximately.

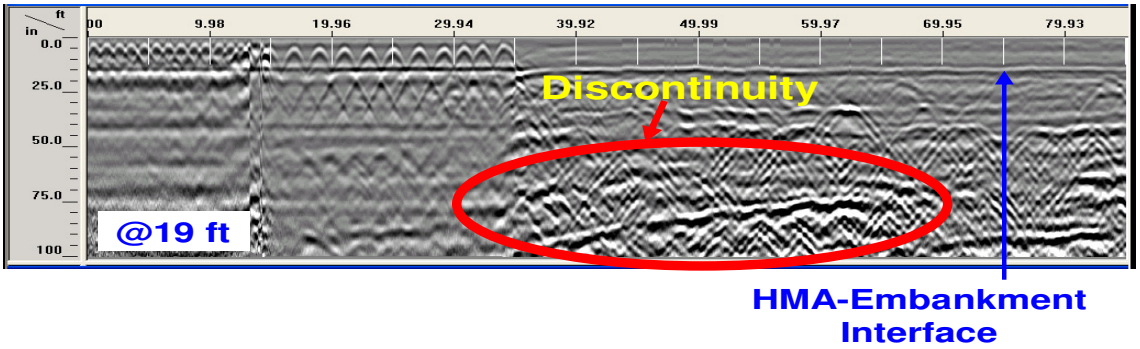


Figure 3.36 Typical linescan of west approach slab with Medium Frequency Antenna

From the GPR information, a core plan was established for the two approach slabs, presented in Figure 3.37. Six cores per slab were identified on intact and anomalous areas, as detailed in Figure 3.37. On the west slab, cores were designated from W1 to W2 and on the east slab from E1 to E6. From these six cores, four were marked on the approach slab and two on the asphalt pavement.

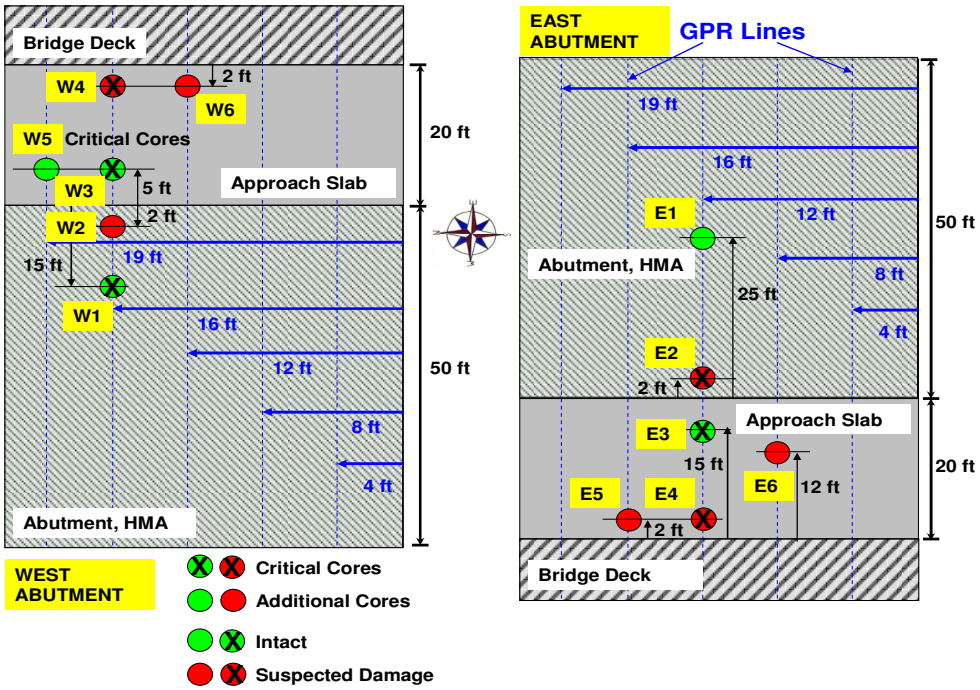


Figure 3.37 Proposed Core layout for San Antonio Bridge

Detailed views from some of the cores retrieved on the west and east approach slabs are presented in Figures 3.38 and 3.39 respectively. Cores W1 and W2 (on the HMA ramp) were about 18 in. long. Core W1 was cracked at depths of 6.5 and 12 in., while core W2 showed severe stripping at depths ranging from 10 to 15 in. On the approach slab, cores W3 to W5 presented a concrete thickness of about 14 in, over a 2 in. thick HMA layer. A 2 to 4 in. void was observed underneath the HMA. On the east approach slab, core E2 had a thickness of 18 in. with no indications of damage. Core E3 had a concrete layer of 13 in. that cracked at about 3 in. from the top, had a void of about 1 to 2 in. and a layer of stripped asphalt about 4 in. thick.

Core E4 was intact and was 14 in. thick, with a void of about 5 in. appeared with some stripped asphalt underneath. Core E5 was 12.5 in. thick that had some stripped asphalt as well. Core E6 was not feasible to obtain. The summary of results from the twelve cores retrieved and the comparison with the GPR results is presented in Table 3.4. Estimated depths of bottom

layers of PCC or HMA are highlighted. In general GPR depth estimation compared favorably well with ground-truth data, since measured and estimated depths only differed by less than 1 in. The only outlier was core W5, where estimated depth was about 12.1 in., and the actual depth from retrieved core was 14 in.

Figure 3.38 Retrieved cores from west slab

Figure 3.39 Retrieved cores from east slab of San Antonio Bridge

In addition to cores, Dynamic Cone Penetrometer tests (DCP) were carried out on the embankment at some of the core locations. As an example results from core W2 (on west HMA ramp) and core E4 (on east approach slab) are presented in Figure 3.40. The accumulated blow counts and the corresponding moduli values versus depth are shown. In both cases two different soil layers were identified. For core W2, an interface seems to appear at a depth of 22 in., and for E4, this interface materializes approximately at 35 in. This top layer in both cases is

softer than the bottom. Similar observations were obtained for all DCP tests conducted. Overall results are summarized in Table 3.5.

Table 3.4 Comparison of retrieved cores with GPR data

Core Designation	Location	Core Results	Interfaces from High Frequency GPR Data
W1	HMA ramp, at 16 ft from south edge, 2 ft from approach slab joint	18 in. of ACP over Base/embankment	1.77, 3.46, 5.40, 8.10, 12.30, 17.70
W2	HMA ramp, at 16 ft from south edge, 15 ft from approach slab joint	17 in. of ACP over Base/embankment	1.86, 3.54, 5.74, 8.40, 11.30, 17.38
W3	Approach slab, at 16 ft from south edge, 5 ft from approach slab joint	13 in. of PCC, 2 in. of Void/AC	4.47, 12.74, 16.96
W4	Approach slab, at 16 ft from south edge, 2 ft from bridge joint	14 in. of PCC, 2 in. of AC/ 3 in. of Voids	4.89, 14.85, 16.70
W5	Approach slab, at 19 ft from south edge, 5 ft from HMA ramp joint	14 in. of PCC, 1 in. of AC/ 4 in. of Voids	3.88, 12.15, 16.20, 17.63, 18.98
W6	Approach slab, at 12 ft from south edge, 2 ft from bridge joint	N/A	3.54, 5.23, 11.73, 13.08, 14.68, 18.98
E1	HMA ramp, at 12 ft from south edge, 25 ft from approach slab joint	N/A	1.94, 2.70, 4.13, 6.83, 9.11, 12.57, 17.63
E2	HMA ramp, at 12 ft from south edge, 2 ft from approach slab joint	17.5 in. of ACP	2.45, 4.98, 8.44, 12.49, 17.72
E3	Approach slab, at 12 ft from south edge, 15 ft from bridge joint	13 in. of PCC, 1-2 in. Void, ACP 4 in., stripped materials	2.70, 11.05, 13.58, 16.03, 18.64
E4	Approach slab, at 12 ft from south edge, 2 ft from bridge joint	14 in. of PCC, 5 in. Void, stripped AC	2.62, 13.08, 18.48
E5	Approach slab, at 16 ft from south edge, 2 ft from bridge joint	12.5 in. PCC, 0.5 in. AC, stripped material	3.71, 6.75, 12.15, 13.33, 17.55
E6	Approach slab, at 8 ft from south edge, 12 ft from bridge joint	N/A	3.54, 5.57, 10.71, 11.98, 15.69, 18.39

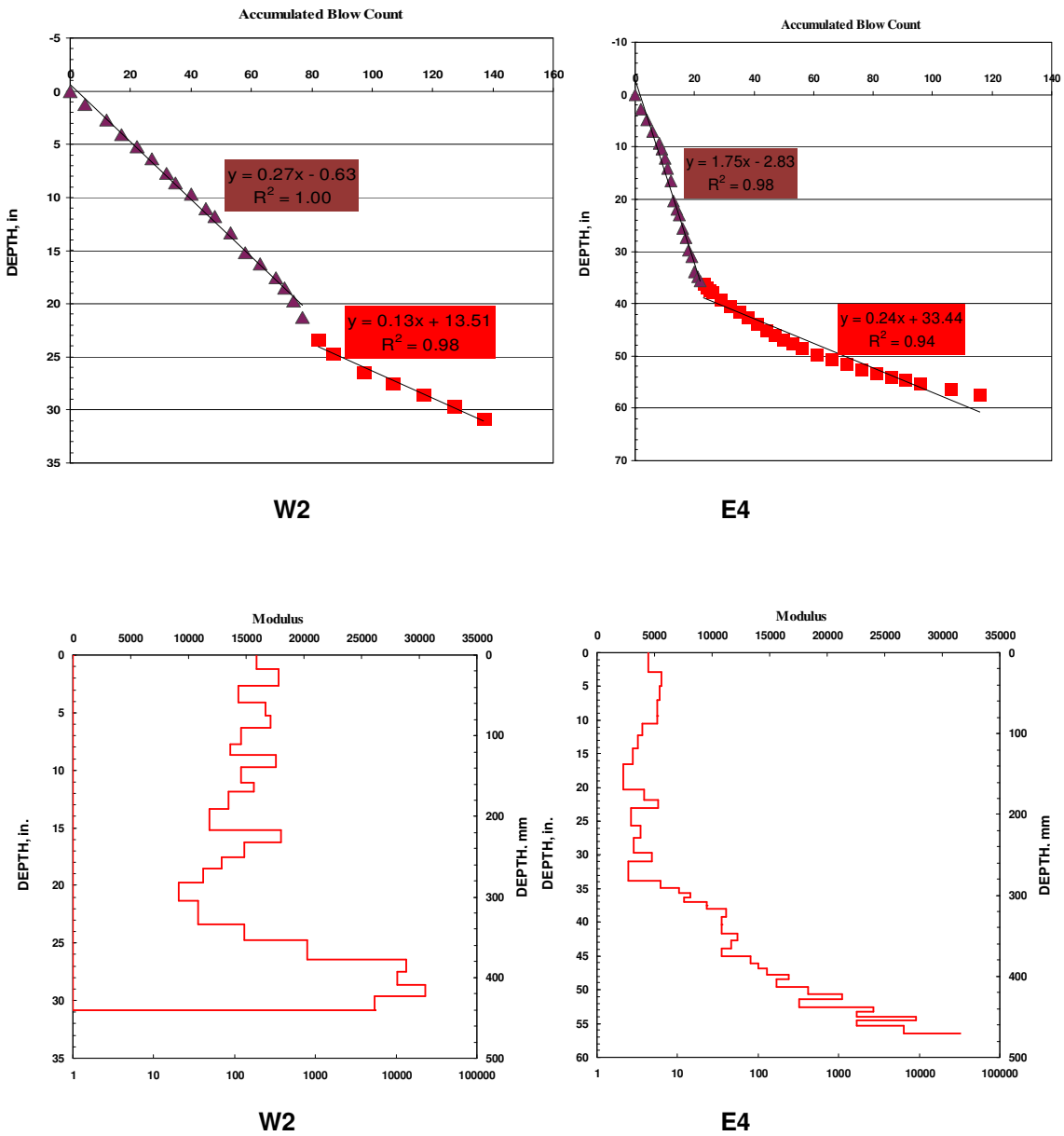


Figure 3.40 DCP sample results

Table 3.5 Summary of DCP results

DCP Location	Slope Layer 1	Slope Layer 2	Approximate Interface Depth (in.)	Initial Modulus (ksi)	Modulus at Interface (ksi)	Final Modulus (ksi)
W1	0.22	0.08	25	15	11	37
W2	0.27	0.13	22	16	9	30
W3	0.21	0.39	6	23	14	23
W4	0.87	0.80	8	11	4	7
W5	1.73	0.24	65	10	3	23
W6	N/A					
E1	N/A					
E2	0.43	0.17	27	21	7	22
E3	0.31	0.21	17-26	11	4	22
E4	1.75	0.24	35	5	3	31
E5	0.27	0.18	20	18	11	21
E6	N/A					

In general, the slope of layer 2 is more horizontal than layer 1, indicating a stiffer layer when depth increases. However, for W3 the slope value for layer 2 is larger, but in this case the interface appears to be at a very shallow depth of 6 in. For W4, the interface also manifests close to the top of the soil layer (8 in.) and both slopes have very similar values. For the rest of the tests on the west slab, cores on the HMA ramp (W1 and W2) had an interface of about 20 in. and for the case of W5, it appeared at 65 in. On the east slab, approximate soil interface was between 17 to 35 in. and for E3; it is not clear since a void was found between 17 and 26 in. Initial modulus widely ranged between 5 and 23, decreased always with depth until the interface was more or less reached and then increased to a value larger than the initial for all cases except W4.

Potential Void Volume Estimation under Approach Slabs

Similar to the Quanah site, an estimation of the suspected voids under the approach slabs was carried out. As an example, Figure 3.41 shows a linescan of the west approach slab and at 16 ft from the south edge of the slab. In this case, the pavement structure comprised of a concrete slab on top of a HMA layer and the embankment underneath. The bottom of the

approach slab, bottom of the HMA-embankment interface, and the suspected void interface were identified at several selected points. With the assumed dielectric constant, estimated depths were obtained and are presented in Figure 3.42. The suspected void under the approach slab adjacent to the bridge joint is also depicted. This suspected voided area was measured by calculating the compound average difference of the bottom of the approach slab and the suspected void at selected locations. For the case shown in Figure3.42, the voided area extends about 8 ft from the bridge joint with a somehow triangular shape. The estimated voided area for this location was about 1.30 ft^2.

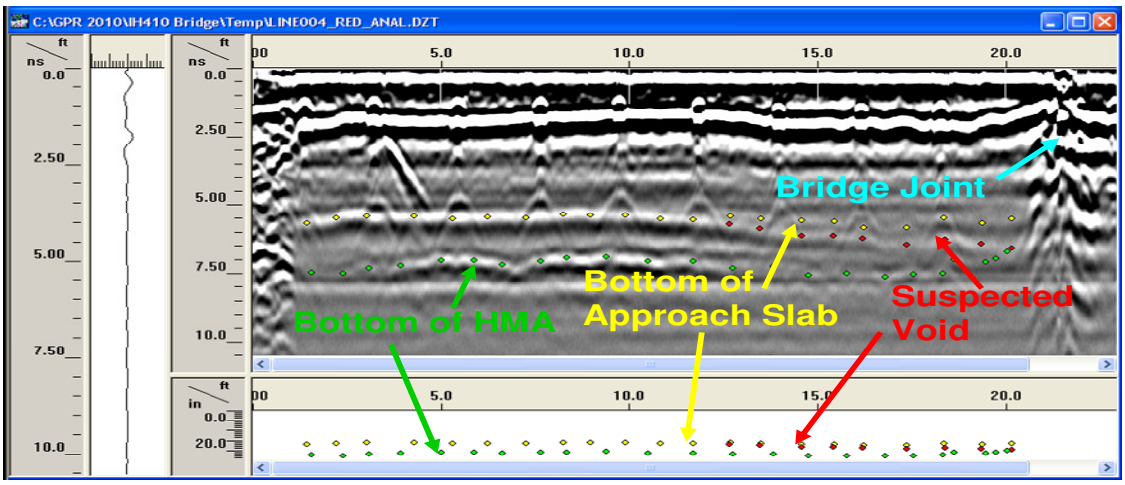


Figure 3.41 Selection of points from GPR linescan

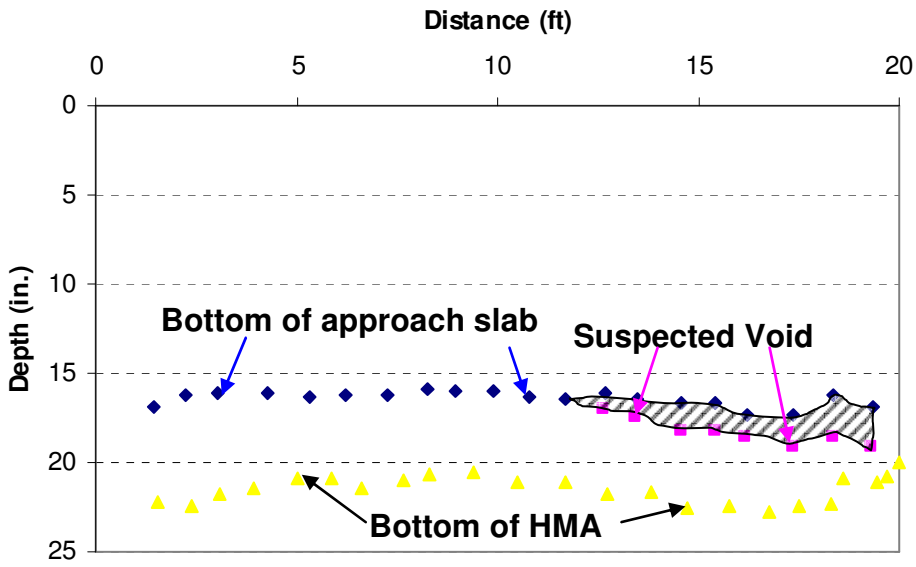


Figure 3.42 Estimated depth profile

The same process was implemented on the rest of the four longitudinal lines obtained for each slab (see Figure 3.33 for details). Results for the west slab are summarized in Table 3.6. It can be observed that the region at 19 ft from the south edge seems to have largest voided area (2.02 ft2) and the smallest was obtained at 12 ft (0.44 ft2). Moreover, the estimated void volume was obtained by measuring the average of the voided area at the five locations and multiplying this area by the width of the lane (24 ft). For this slab, the approximate void volume was 27 ft3. Similarly, the same procedure was conducted for the east approach slab and the estimated void areas and volume are presented on Table 3.7. For all locations in the east slab, the voided area was about 2 ft2 and the final volume was 53 ft3, twice the volume than on the west slab.

Table 3.6 Void area and volume under west approach slab

	West Slab				
Distance from South Edge (ft)	**@4ft**	**@8ft**	**@12ft**	**@16ft**	**@19ft**
Void area (ft2)	**0.87**	**0.93**	**0.44**	**1.30**	**2.02**
Void Volume (ft3)	**26.7**				

Table 3.7 Void area and volume under east approach slab

	East Slab				
Distance from South Edge (ft)	**@4ft**	**@8ft**	**@12ft**	**@16ft**	**@19ft**
Void area (ft2)	**2.30**	**1.53**	**2.69**	**2.39**	**2.11**
Void Volume (ft3)	**52.9**				

Based on the estimated volume of voids, use of Flowable fills is proposed as a suitable mitigation method. However the proposed solution was not implemented due to the uncertainty in the long term performance of the approach slabs repaired using this technique. The distressed bridge on IH 410 in San Antonio is yet to be repaired.

3.3 Summary

This chapter mainly presented the maintenance measures that are either used or considered as remedial methods after the approach settlement problems are detected. A brief introduction to the need and importance of adopting suitable remedial measures to mitigate the settlement related problems in distressed bridge approach slabs is presented.

The purpose of this chapter is to provide appropriate mitigation techniques that can be investigated to treat the distressed bridges. Hence four bridge approach slabs with settlement related problems are identified. The location of the bridges, the corrective methods employed, procedure of the method used and the geophysical studies performed on the bridges along with recommendations are discussed. The corrective methods both proposed and recommended include Polyurethane injection, Soil nailing, Flowable fill and the use of light weight Geofoam blocks. Among these methods, polyurethane injection has provided some enhancements. However, void estimation is needed prior to selecting this method. Also, a few quality control

measures need to be established and implemented when using this method. In the case of soil nailing, the method was mainly used to enhance the stability of the MSE wall setup, which in turn can control erosion thereby enhancing the stability of the whole bridge including the approach slabs at both ends.

Two other bridges in Childress and San Antonio districts were also studied for potential implementation of light weight Geofoam blocks and use of Flowable fills. Both methods are considered to have high potential to mitigate settlements. However, these methods are not explored further and they need to be investigated in future studies.

CHAPTER 4

MITIGATION OF SETTLEMNTS USING EXPANDED CLAY AND SHALE (ECS) AGGREGATE EMBANKMENT FILL SYSTEM

4.1 Introduction

This chapter presents the results of numerical modeling performed in the development of design charts for Expanded Clay and Shale (ECS) aggregate fill embankments. The ECS was used as a backfill material to construct the northbound lanes of a new bridge structure constructed at south end extension of SH-360. The results for the properties of soil used as input parameters in the modeling were obtained from previous studies by Archeewa (2010). Also the field monitored data used to validate the modeling analysis was obtained from the studies by Archeewa (2010). The modeling was extended to other embankment configurations by varying the heights and slopes of the embankment, thicknesses of the subgrade soil and also the compression indices of the subgrade soil after validating the section with the measured data for the development of design charts.

4.2 Construction of the Embankment with ECS

The light weight aggregate fill, expanded clay shale or ECS was used to construct the embankment of the bridge on the highway SH360 in Arlington, Texas. The ECS was delivered to the bridge site in trucks like natural fill materials. Then, it was spread uniformly in horizontal by tracked vehicles in layers not exceed 12 in. thick. Each layer was compacted using vibratory compaction equipment with 10 tons static weight. A minimum of 3 passes is required to ensure full compaction. The best method of site control is by measuring the settlement of the compacted layer, which should not be more than 10% of the layer thickness or 1.2 in. Totally, a quantity of 26, 242 yd^3 was used in the embankment construction in the project.

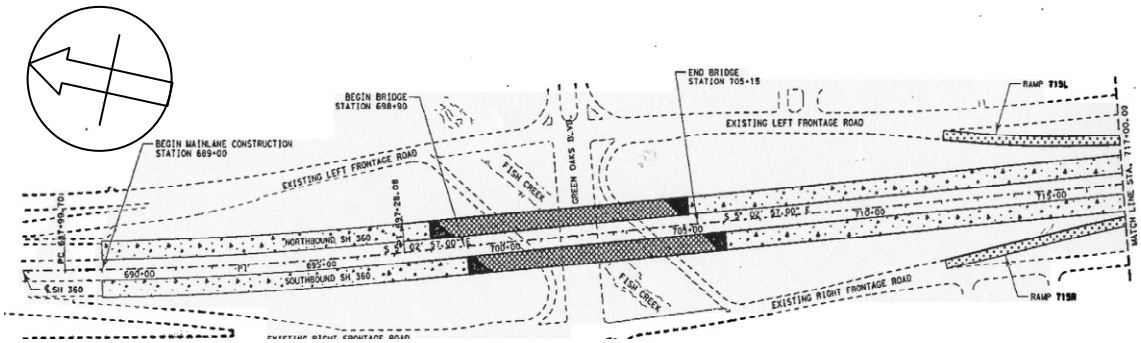


Figure 4.1 Details of ECS Bridge Site, SH 360, Arlington, TX

4.3 Finite Element Method

The Finite element method (FEM) has been found to be the most powerful numerical techniques for solving problems in the mechanics of continuous media (Bugrov, 1975). Nowadays, the FEM plays an important role in all branches of engineering for the analysis and design of the structures (Bathe, 2003). The analysis is typically performed by transforming the physical problem, an actual structure and structural components, into a mathematical model (Bathe, 2003). By using a series of algebraic equations, the numerical models can be solved, and the quantities of interested parameters for example, stress, strain and deformation at the points of interest can be approximately obtained (Burd, 2004)

The FEM is generally consisted of nodes and elements to form a finite element mesh of the structures. A typical two dimensional mesh with 6 nodes is shown in Figure 4.2.

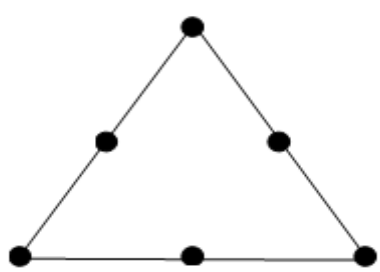


Figure 4.2 Six noded triangular element (Plaxis manual)

The nodes are not only the points where more then one or two elements connect to the others, but also the points where values of the primary variable of interested parameter are calculated (Burd, 2004). In the FEM analysis, the values of the stain will be calculated and then interpolated in for the entire structure. Later, a relationship between stress-strain of a material behavior usually termed as a constitutive law is used to calculate the stress occurred in the mesh. Last, the force acting on each node obtained from the previous step will be calculated further to compute the nodal displacements by relating the nodal forces with the stiffness equations.

For the one dimensional (1-D) consolidation, the phenomenon can be described by the following differential equation:

$$\frac{\partial p}{\partial t} = c_v \frac{\partial^2 p}{\partial z^2} \tag{4.1}$$

Where

$$c_v = \frac{kE_{OED}}{\gamma_w}$$

$$E_{OED} = \frac{(1-v)E}{(1+v)(1-2v)}$$

$$Z = H - y$$

k is permeability of soil

E is the Young's modulus

υ is the Poisson's ratio

γ_w is unit weight of water

E_{OED} is the oedometer modulus

The analytical solution for the above Equation 4.1 in a relation to p/p_0 as a function of time and position is presented by Verruijt (1983) and this equation is presented in the following:

$$\frac{p}{p_o}(z,t) = \frac{4}{\Pi}\sum_{j=1}^{\infty}\frac{(-1)^{j-1}}{2j-1}\cos\left[(2j-1)\frac{\Pi}{2}\frac{y}{H}\right]e^{\left[-(2j-1)^2\frac{\Pi^2}{4}\frac{c_v t}{H^2}\right]} \tag{4.2}$$

The results of both numerical and analytical method are presented in Figure 4.3. The dotted lines are the results from analytical method and the continuous lines are the results from numerical analysis. It can be seen that the results from both methods are close.

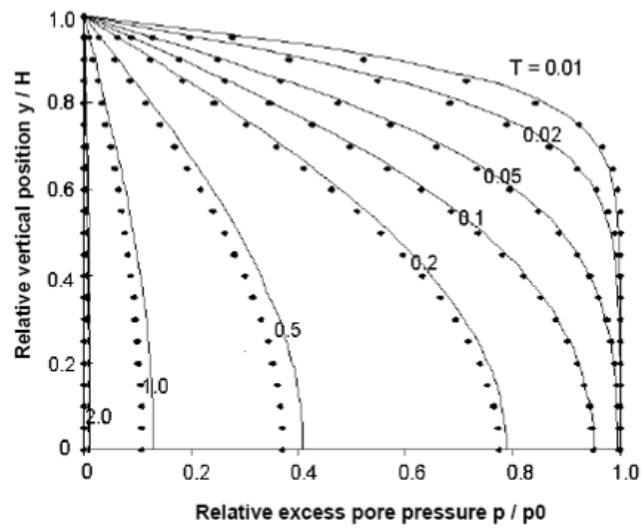


Figure 4.3 The results of excess pore pressure as a function of height from Numerical and analytical methods (Plaxis Manual)

4.4 Modeling of Light Weight Embankment System

4.4.1 Geometry and boundary conditions of the test section

A cross-section and subsurface profile of the light weight fill material, expanded clay shale or ECS aggregate filled embankment is shown in Figure 4.4. The embankment has a total height of 30 ft (9 m) from an existing ground and a side slope of 2H: 1V. The embankment was constructed on a soft clay layer with a thickness of 16 ft (5 m), which is underlain by 10 ft (3 m) sand layer.

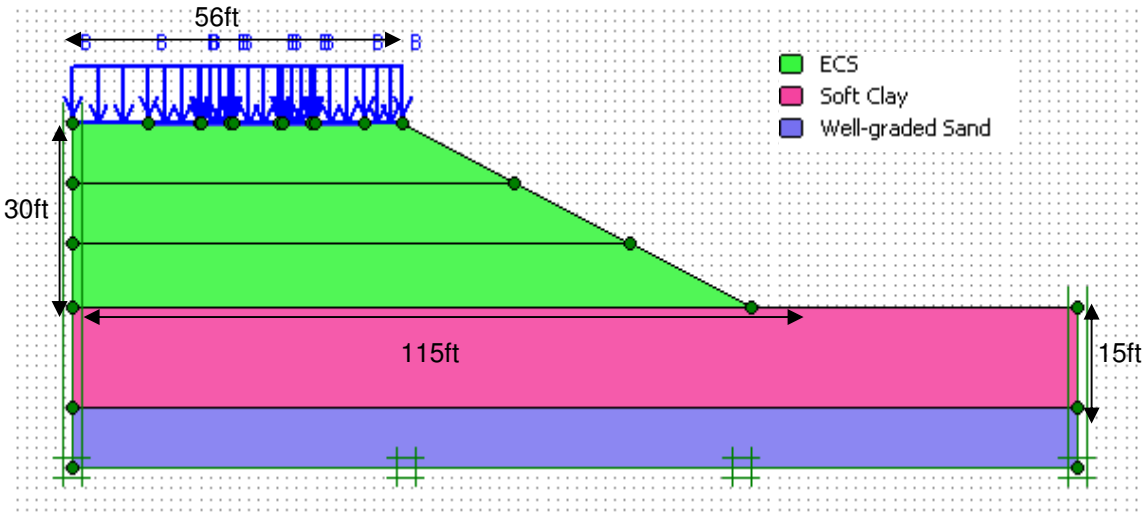


Figure 4.4 Geometry and boundary conditions of the ECS test section

4.4.2 Material property values in a numerical analysis

In this model analyses, a soft soil model is used to simulate soft clay material, while a Mohr Coulomb model is used to simulate ECS and well-graded sand materials. Most of the strength parameters are derived from the laboratory study results obtained from previous studies by Archeewa (2010).

The analyses were performed to study the settlement behavior in a long-term duration due to a consolidation phenomenon. Therefore, the settlements occurred during the construction phases are disregarded. The long-term settlement analyses are performed with a consolidation based model, and are carried out until the ultimate pore pressure state is reached, or in other words, until the dissipation of excess pore water pressure. The displacement of the boundary at x=0, y=0 was restricted in all directions as well as the boundaries at the points of the base. The material properties used in the analyses are given in Table 4.1

Table 4.1 Properties and model type of the materials used in the test section model analysis

	Unit	ECS	Soft clay	Well-graded Sand
Model type		Mohr-Coulomb	Soft soil	Mohr-Coulomb
Moist density, γ_m	pcf	39.8	91	96.7
Sat. density, γ_s	pcf	50	109.5	109.9
Elastic modulus, E^{50}_{ref}	psf	$1x10^8$	-	$3.8x10^8$
Poisson's Ratio	-	0.15	-	0.15
Cohesion, c	psf	1570	940	0
Friction Angle, φ	°	49.5	5	33
Permeability, k	ft/min	$2x10^{-3}$	$2x10^{-8}$	$1.2x10^{-4}$
Compression Index, C_c	-	-	0.34	-
Recompression Index, C_r	-	-	0.023	-
Over Consolidation Ratio, OCR	-	-	3	-
Initial void ratio, e_o	-	-	0.80	-

4.4.3 Discretization of the test section

A two-dimensional plain-strain model with 15-node triangular elements is used to model the test embankment as shown in Figure 4.5. The embankment in the model consists of 3 types of materials, ECS, clay and sand. In Figure 4.5, the highway pavement is also seen on the top of the embankment model, as a blue line, which has a length equal to the width of the pavement in the field.

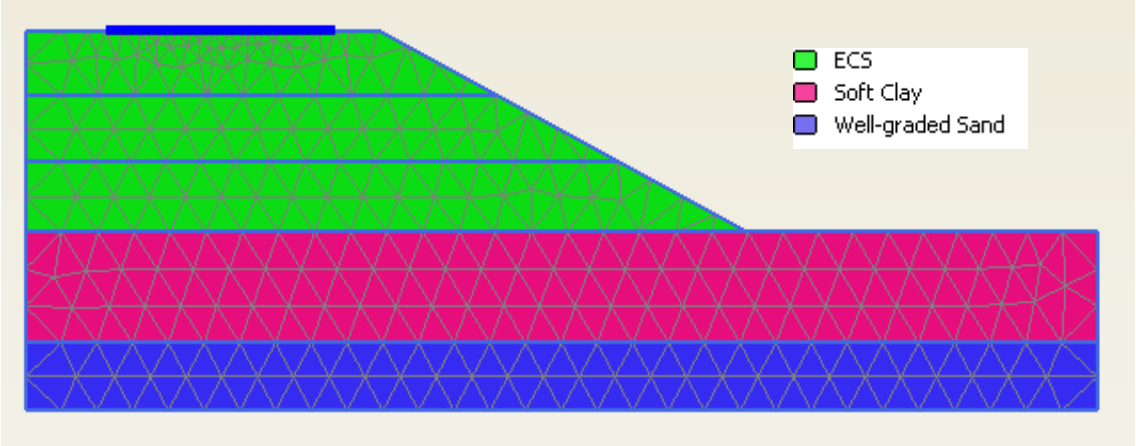


Figure 4.5 Nodes and elements in the test section

4.4.3.1 Settlement Analysis

The total height of the embankment of 30 ft (9 m) was divided into 3 layers to simulate embankment construction phases in the analysis as shown in Figure 4.5. Figure 4.6 presents the construction phases in the numerical analyses. It can be seen from the figure that duration of the embankment construction for each layer was 25 days, and there after, the embankment was left to be consolidated for another 25 days in the second stage. In the initial stage, the soil self weight was used as a parameter that induced soil stress.

In the second stage, each layer in the embankment is modeled for consolidation. The embankment construction is simulated in that way until the whole embankment construction is completed. The simulation is done such that it replicates the compaction effort performed in real practice. After the completion of the last layer of the embankment, the whole embankment is compacted by applying a uniform load. The highway pavement is constructed after the compaction of the embankment and the calculation of the settlement was reset to zero. After the completion of the highway pavement construction, traffic load is applied. It should be noted that the model analyzed by using a gravity load of the material and the traffic load of 80 kN/m2.

Since only elevation surveys and vertical inclinometer monitoring were performed at the ECS site at the locations shown on Figure 4.7 (Points A and B). Therefore, those two points are selected to investigate the soil movements and to validate the results of the numerical analysis in this study.

Identification	Phase no.	Start from	Calculation	Loading input	Time	Stage	Water	First	Last
Initial phase	0	N/A	K0 procedure	Unassigned	0.00 day	L 0	W 0	1	1
Embankment con...	2	0	Consolidation (EPP)	Staged construction	25.00 day	L 2	W 2	2	8
Consolidation of ...	3	2	Consolidation (EPP)	Staged construction	25.00 day	L 3	W 3	9	13
Embankment con...	4	3	Consolidation (EPP)	Staged construction	25.00 day	L 4	W 4	14	36
Consolidation of ...	5	4	Consolidation (EPP)	Staged construction	25.00 day	L 5	W 5	37	62
Embankment con...	6	5	Consolidation (EPP)	Staged construction	25.00 day	L 6	W 6	63	101
Consolidation em...	7	6	Consolidation (EPP)	Staged construction	25.00 day	L 7	W 7	102	142
Embankment com...	14	7	Consolidation (EPP)	Staged construction	50.00 day	L 14	W 14	143	218
Pavement const	8	14	Consolidation (EPP)	Staged construction	70.00 day	L 8	W 8	219	312
Load application	13	8	Consolidation (EPP)	Staged construction	10000.00 ...	L 13	W 13	313	475
Minimum pore pre...	12	13	Consolidation (EPP)	Minimum pore press...	0.00 day	L 13	W 13	476	476

Figure 4.6 Calculation phase in the settlement analyses

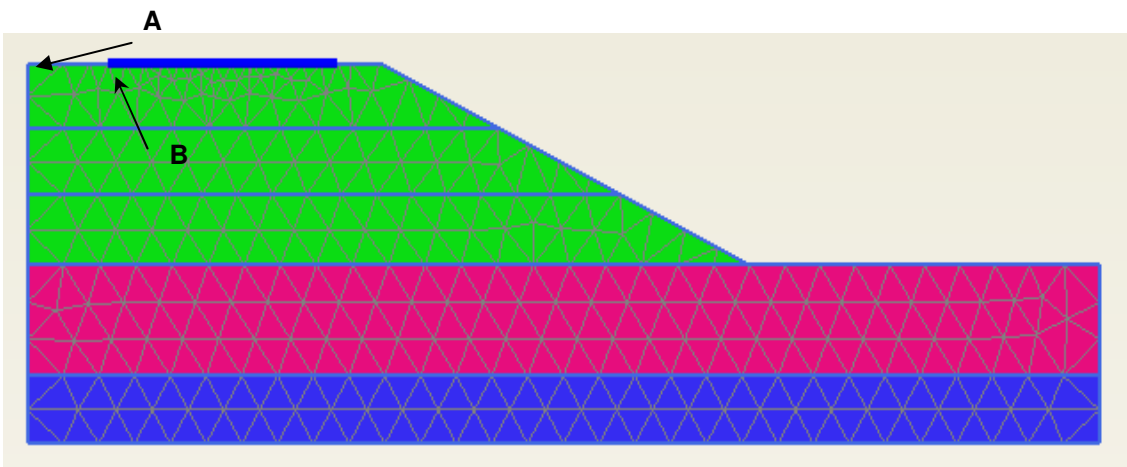


Figure 4.7 Observation points in the settlement calculation

4.4.3.2 Results of the numerical modeling analysis

The results from the FEM analysis are presented in Figures 4.8 – 4.11. Figure 4.8 shows the deformed mesh of the embankment with a displacement magnification scale of 1:50. The maximum long-term displacement in this analysis is equal to 1.66 in. (42 mm) and occurred in the left side of the embankment, which is seen in Figure 4.8. Besides, it can be clearly seen that the soft clay is the layer that experienced the highest amount of the settlement due to the consolidation, which induces the settlement occurred in the ECS embankment.

Figure 4.9 shows the results of total displacements occurred in the embankment. It can be seen in the Figure 4.9 that most of the consolidation occurred in the soft clay layer but the maximum total displacement occurred in the ECS. Figure 4.10 shows the horizontal soil movements occurred in the embankment. Most of the lateral movements occurred at the toe of the slope and on the right side of the embankment, while at the top of the slope and on the left side only small amount of the movements can be noticed. The vertical soil displacements are presented in Figure 4.11. It is obviously seen that the pattern of color shades from the results of vertical movements in Figure 4.11 is quite similar to the pattern from the results of total movements in Figure 4.9, which means the vertical movement is still a predominant factor in this study.

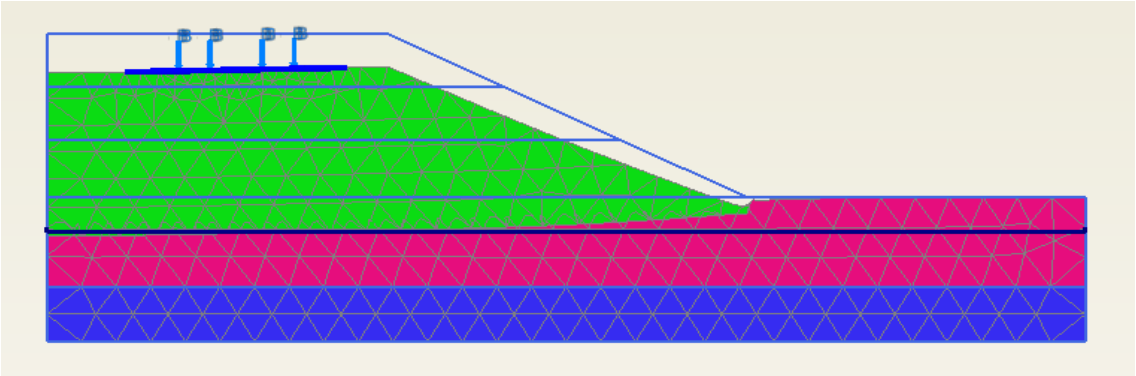


Figure 4.8 Deformed mesh of the test section (displacement scaled up 50 times)

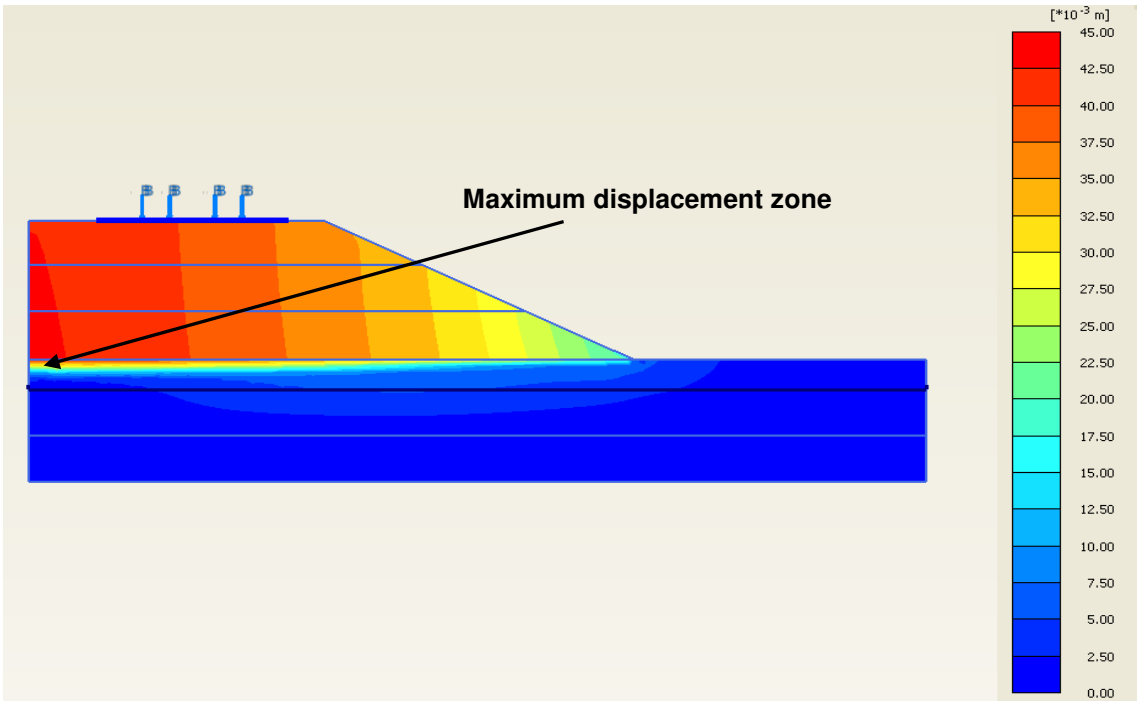


Figure 4.9 Total displacements in the test section

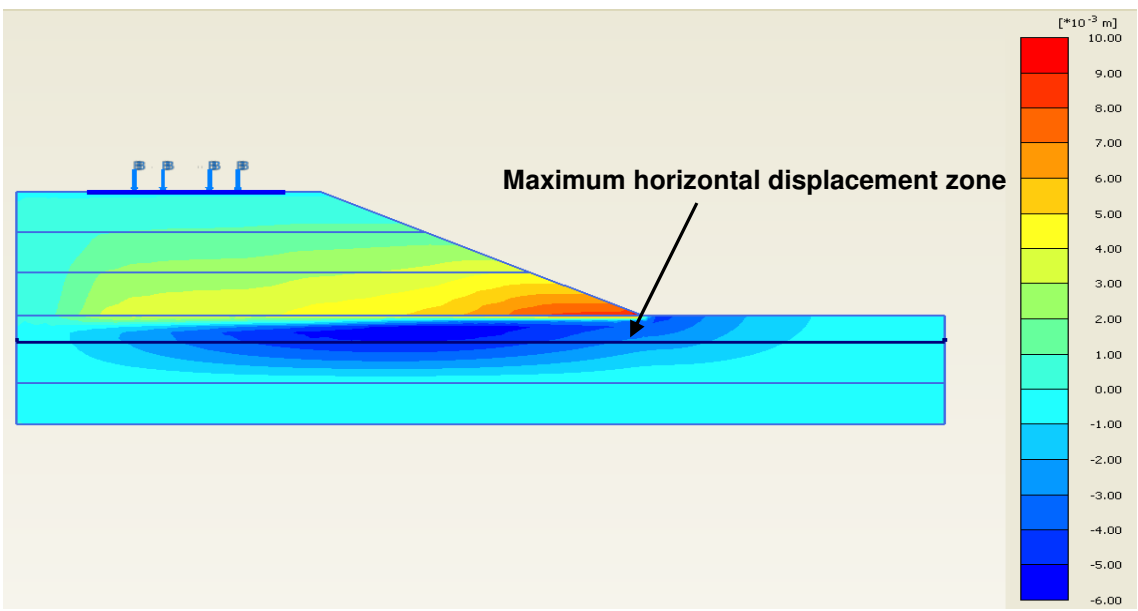


Figure 4.10 Horizontal displacements in the test section

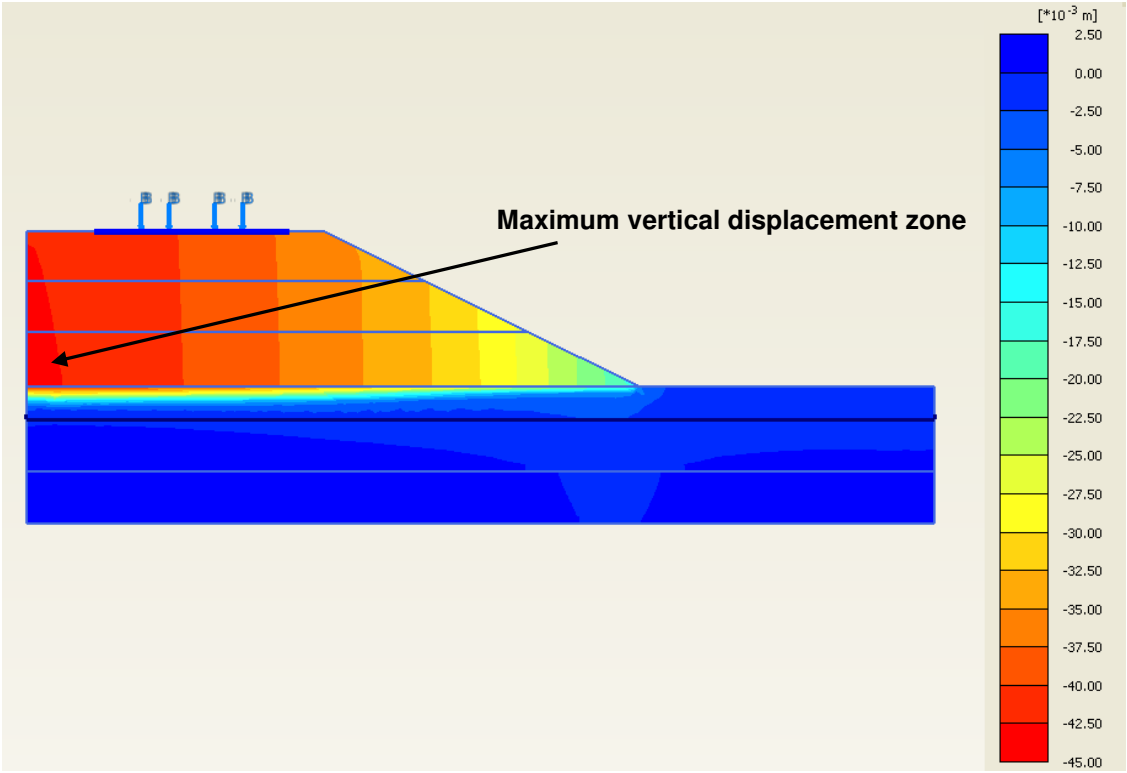


Figure 4.11 Vertical displacements in the test section

4.4.3.3 Model Validation

To validate the parameters used in the model, the results obtained from the above model analysis are used to compare with the monitoring data from the field obtained from previous studies by Archeewa (2010). The comparisons are performed by using data from elevation surveys and vertical inclinometers to investigate vertical soil displacements. The field monitored data of the ECS section obtained from Archeewa (2010) is shown in table 4.2.

Table 4.2 Elevation survey data on Test section site SH 360

Date	Elevation of the interested point (ft)	Soil settlements	
		(ft)	(mm)
16-Jul-06	1.41	0.00	0.00
14-Aug-06	1.39	0.02	6.10
18-Sep-06	1.37	0.04	12.19
14-Oct-06	1.36	0.05	15.24
17-Nov-06	1.35	0.06	18.29
10-Dec-06	1.35	0.06	18.29
15-Jan-07	1.34	0.07	21.34
17-Feb-07	1.34	0.07	21.34
20-Mar-07	1.34	0.07	21.34
13-Apr-07	1.33	0.08	24.38
20-May-07	1.33	0.08	24.38
3-Sep-07	1.33	0.08	24.38
16-Dec-07	1.32	0.09	27.43
18-Jun-08	1.32	0.09	27.43
19-Sep-08	1.31	0.10	30.48
17-Dec-08	1.31	0.10	30.48
18-Jan-09	1.31	0.10	30.48
20-Mar-09	1.31	0.10	30.48
2-Sep-09	1.30	0.11	33.53
6-Nov-09	1.30	0.11	33.53
8-Feb-10	1.30	0.11	33.53
5-Apr-10	1.30	0.11	33.53
14-Apr-10	1.30	0.11	33.53
8-Jun-10	1.29	0.12	36.58

4.4.3.4 Comparison for the Results of Vertical Displacements with Elevation Surveys

The elevation surveys from Table 4.2 were plotted with the results of the numerical analysis as shown in Figure 4.12. It can be seen that the amount of settlements with time from both sources are not indifferent and the prediction showed a good match.

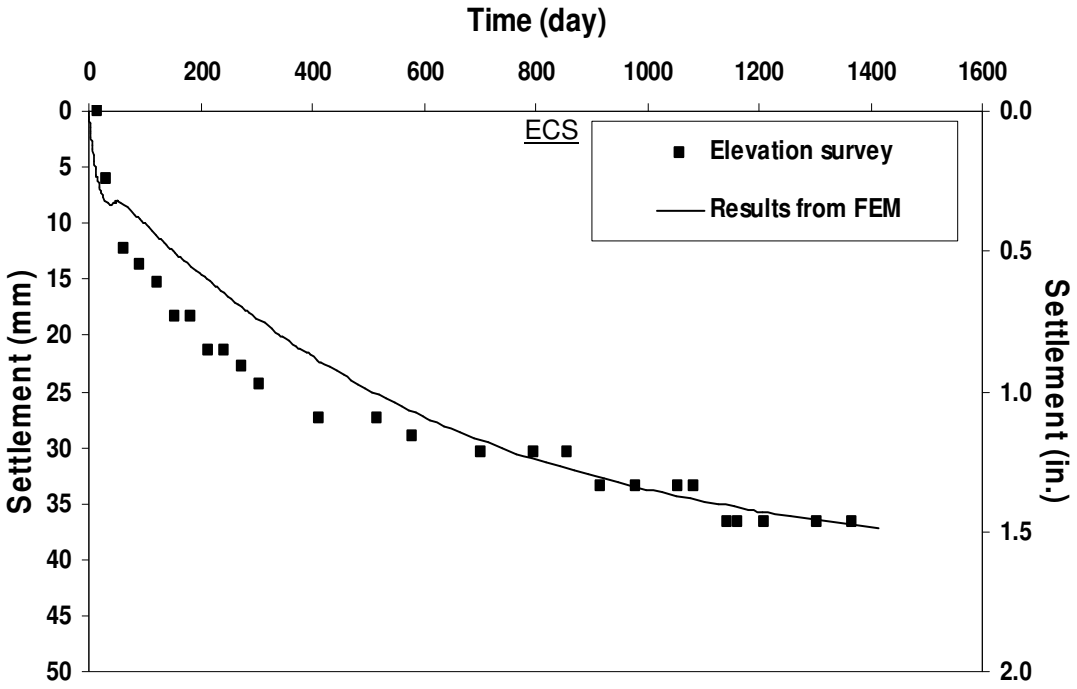


Figure 4.12 Comparison of vertical displacements in a test section between data obtained from elevation surveys and results from numerical analysis

4.4.3.5 Comparison for the Results of Vertical Displacements with Inclinometer Surveys

No data comparison was collected that can be used to perform validation of the lateral soil displacement. From Figure 4.13, it can be noticed that there is no vertical inclinometer casing in the embankment that can provide suitable lateral movement data for model validation. The casings V1 and V2 are located at the locations where the lateral movements can be influenced by different types of embankment fill materials, RAP in a southbound embankment

and ECS in a northbound embankment. The casing V3 and V4 were installed very closed to the end of the embankment, whereas a cross-section in the model is ideally a part of a long continuous embankment. Then, the lateral soil movements monitored from casings V3 and V4 could be data influenced by the boundary conditions. The lateral soil movements measured in the vertical inclinometer casing V1 and V4 are plotted and presented as shown in Figure 4.14 and 4.15, while data from casing V2 are discarded. The reason is the location of the inclinometer casing V2 is too close to the RAP embankment; which the soil movements can be more predominated by RAP than ECS.

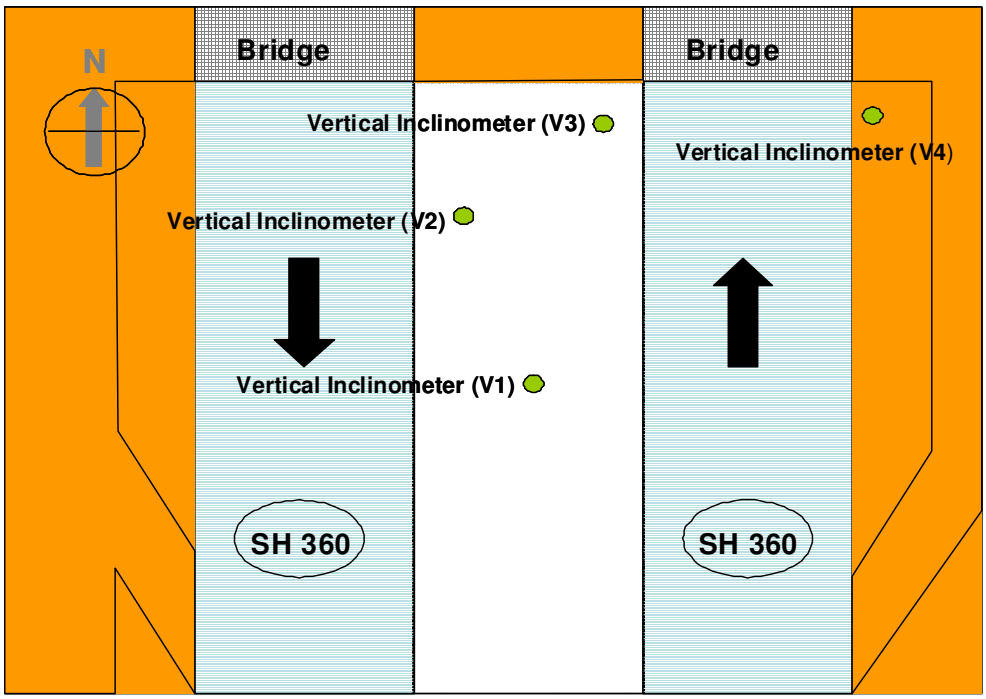
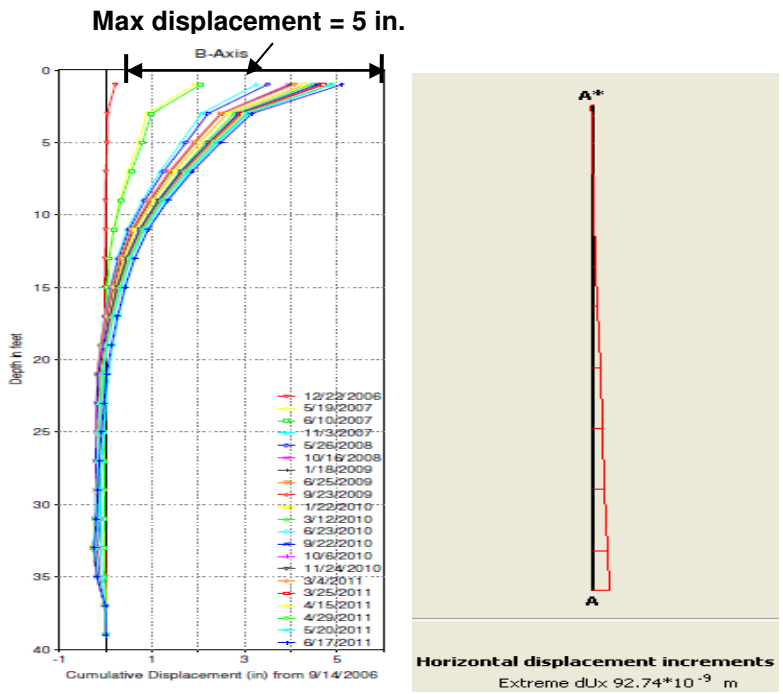


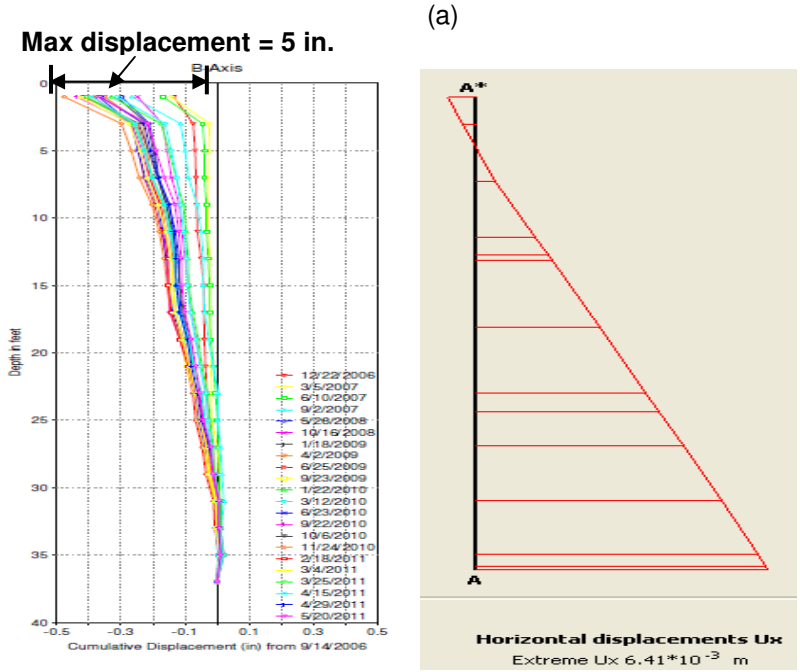
Figure 4.13 Location of vertical inclinometers installed on ECS embankment, SH 360.

Figure 4.14 (a) and (b) reveals the lateral soil movements in the casing V1and V4, respectively. In Figure 4.14 (a), only the movements along the embankment cross-section (B-axis) were presented here, since the movement in A-axis cannot be modeled in this analysis. It

can be seen in Figure 4.14 (a) that the lateral displacements monitored from the casing V1 move toward the left, while the results from the analysis are extremely small having a displacement pattern different from the monitored data. The displacements recorded from the casing located in the middle of the highway median are very small which are close to the results from the numerical analysis. Figure 4.14 (b) shows the lateral soil displacements in a direction of the cross-section of the embankment. It can be seen that the displacement profile from monitored data has a large displacement at the top and a low value at the bottom of the embankment, which is different from the displacement profile resulting from the numerical analysis. This can be an evidence that apart from the consolidation phenomenon, the lateral movements occurred in the field could come from another reason like soil erosion as suspected in the previous chapter. However, the soil erosion phenomenon is very complicated and not able to be modeled in the Plaxis. It should be noted also that the monitored lateral soil movements from casing V3 are not presented, because mostly the soil movements were influenced from the movement happened around the case V4 and are too complex for the analysis.



(a)



(b)

Figure 4.14 Comparisons of lateral soil movements between monitored data and numerical results in the vertical inclinometers (a) V1and (b) V4

4.4.4 Control Embankment Section

The control section is an embankment section constructed with local fill and this section was located on the north side of the bridge. The settlement investigations using both elevation surveys in the field and the numerical analysis are performed, which are used further to compare with the values from the ECS embankment test section to evaluate how ECS material can mitigate the settlement problem that can occur in the bridge embankment section. The geometry, boundary conditions and discretization of the control section are the same as the ones used in the ECS test section by replacing the embankment fill material of ECS with sandy clay fill material as shown in Figure 4.15.

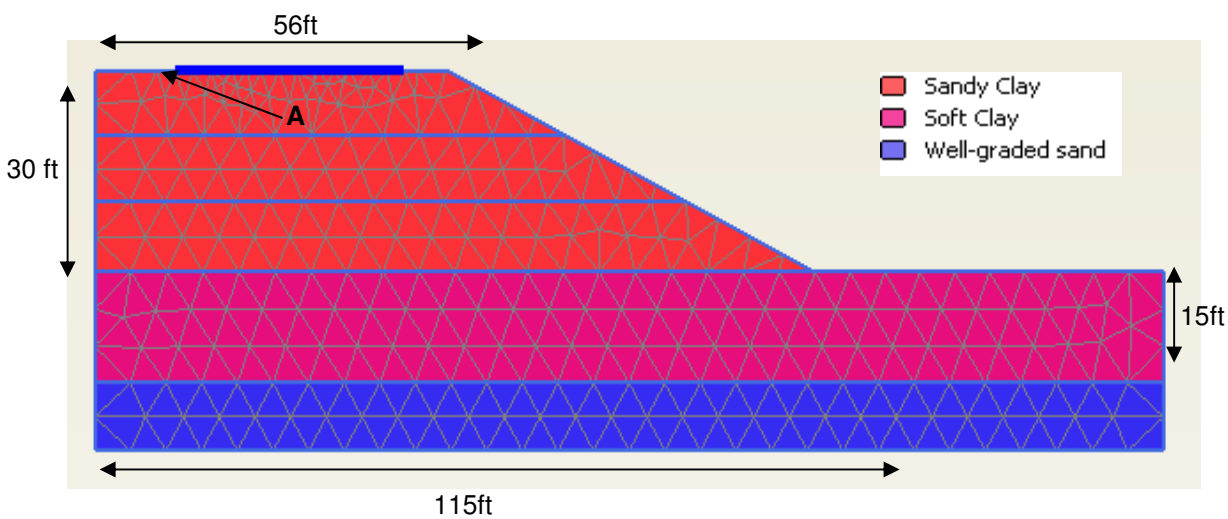


Figure 4.15 Nodes and elements in the control section

4.4.4.1 Settlement Analysis

The settlement numerical analyses on the control section at Point A are performed in the same way as was done on the test section. The total embankment of 30 ft (9 m) high was divided into 3 layers with the same duration of the construction phases, load multiplier factor,

and traffic load as in the test section analyses. Material properties used in the analyses are given in Table 4.3.

4.4.4.2 Results of Numerical Analysis

Figures 4.16 – 4.19 show the results from numerical analysis in the control section. Figure 4.16 presents the deformed mesh of the embankment with a displacement scale enlarged to 20 times. It can be seen that the maximum settlement of the embankment of 0.75 in. (0.19 m) occurred evenly in an area under the embankment crest. The results of the total displacements occurred in the embankment are presented in Figure 4.17. The figure reveals that consolidation mostly occurred in the subgrade layer and that consolidation also resulted in displacements within the embankment. Figures 4.18 and 4.19 show the lateral and vertical soil movements occurred in the control embankment, respectively. It can be seen that the vertical movement is still a predominant factor in the control embankment section.

Table 4.3 Properties and model type of the materials used in control section model analysis

	Unit	Sandy clay	Soft clay	Well-graded Sand
Model type		Soft soil	Soft soil	Mohr-Coulomb
Moist density, γ_m	pcf	94.2	89.3	96.7
Sat. density, γ_s	pcf	109.5	107.4	109.9
Elastic modulus, E^{50}_{ref}	psf	-	-	3.8×10^{8}
Poisson's Ratio	-	-	-	0.15
Cohesion, c	psf	420	940	0
Friction Angle, φ	°	18	5	33
Permeability, k	ft/min	1×10^{-6}	2×10^{-8}	1.2×10^{-4}
Compression Index, C_c	-	0.12	0.34	-
Recompression Index, C_r	-	.030	0.023	-
Over Consolidation Ratio, OCR	-	3	3	-
Initial void ratio, e_o	-	0.55	0.80	-

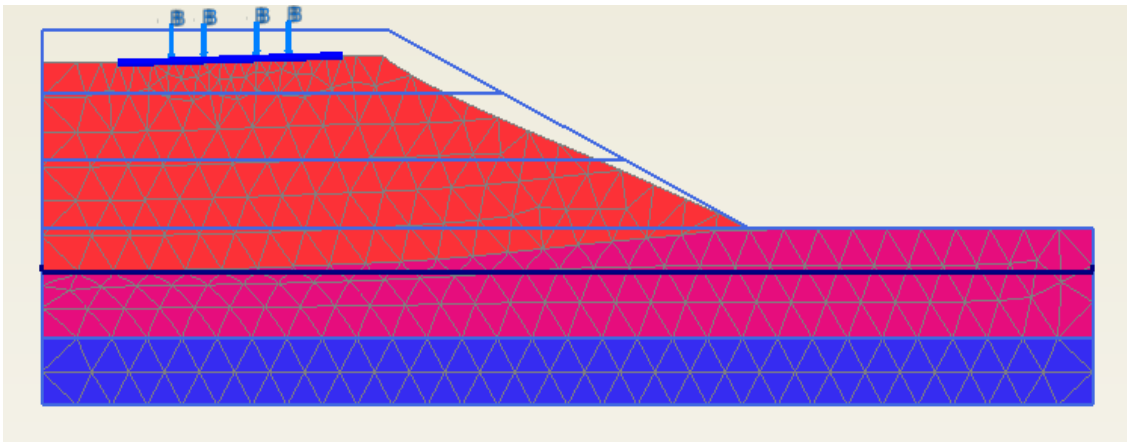


Figure 4.16 Deformed mesh of the control section (displacement scaled up 20 times)

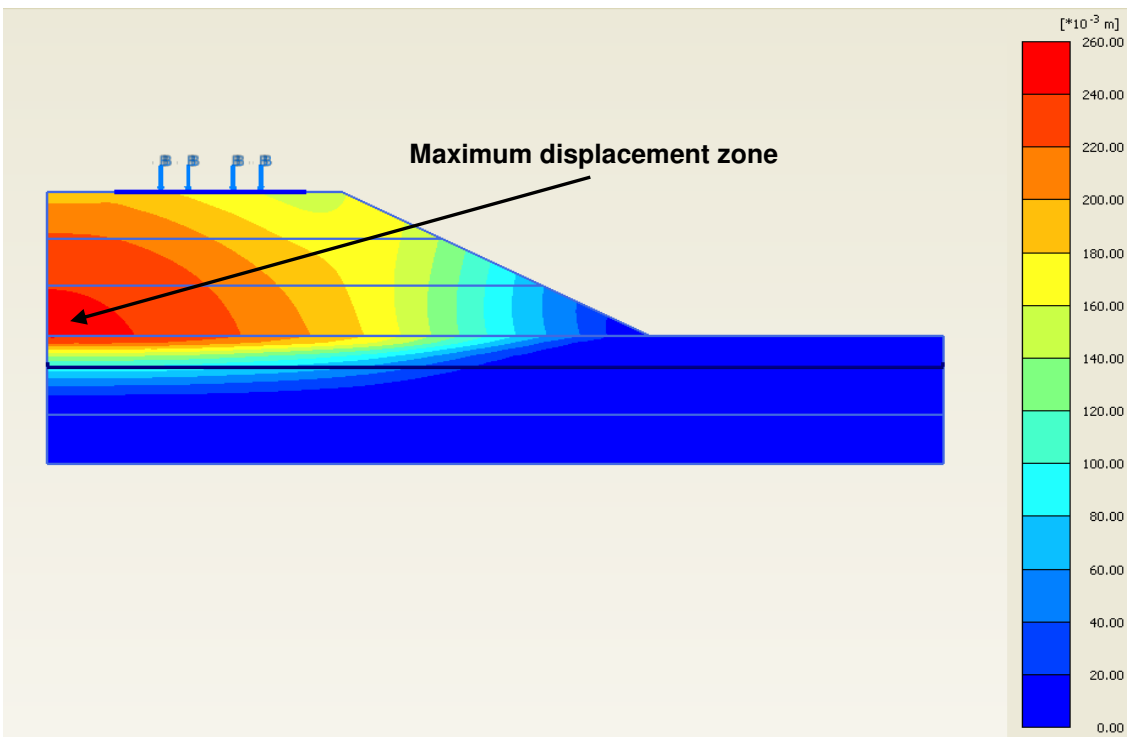


Figure 4.17 Total displacements in the control section

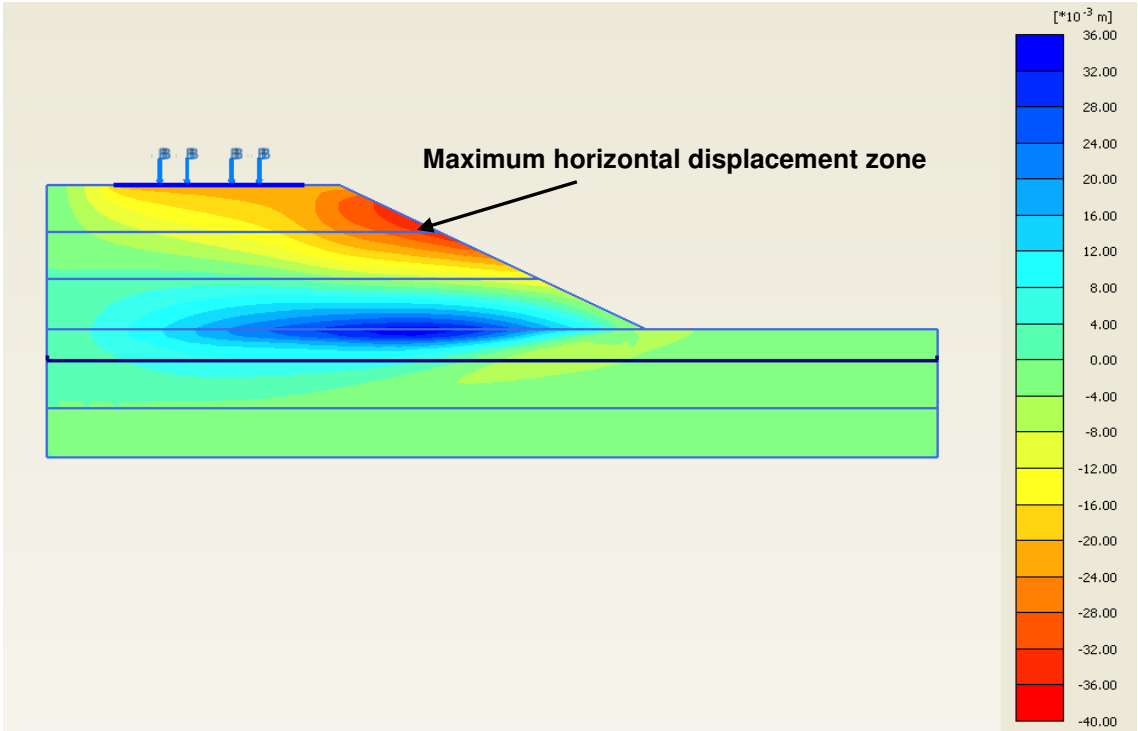


Figure 4.18 Horizontal displacements in the control section

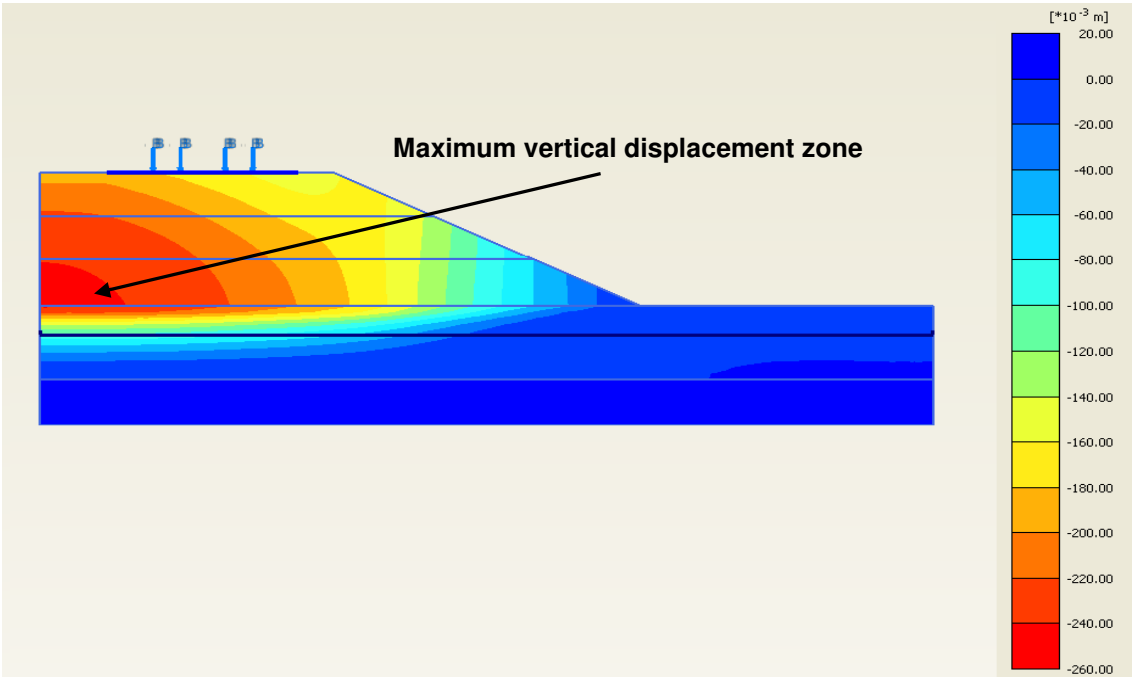


Figure 4.19 Vertical displacements in the control section

4.4.4.3 Model Validation

To validate the parameters used in the model, the results obtained from the above model analysis are used to compare with the monitoring data from the field obtained from previous studies by Archeewa (2010). The comparisons are performed by using data from elevation surveys and vertical inclinometers to investigate vertical soil displacements. The field monitored data of the control section obtained from Archeewa (2010) is shown in table 4.4.

Table 4.4 Elevation survey data on the control section site SH 360

Date	Elevation of the interested point (ft)	Soil settlements	
		(ft)	(mm)
16-Jul-06	-6.98	0.00	0.00
14-Aug-06	-7.00	0.02	6.10
18-Sep-06	-7.01	0.03	9.14
14-Oct-06	-7.03	0.05	15.24
17-Nov-06	-7.05	0.07	21.34
10-Dec-06	-7.06	0.08	24.38
15-Jan-07	-7.07	0.09	27.43
17-Feb-07	-7.08	0.10	30.48
20-Mar-07	-7.09	0.11	33.53
13-Apr-07	-7.10	0.12	36.58
20-May-07	-7.11	0.13	39.62
3-Sep-07	-7.14	0.16	48.77
16-Dec-07	-7.15	0.17	51.82
18-Jun-08	-7.16	0.18	54.86
19-Sep-08	-7.18	0.20	60.96
17-Dec-08	-7.19	0.21	64.01
18-Jan-09	-7.22	0.24	73.15
20-Mar-09	-7.22	0.24	73.15
2-Sep-09	-7.23	0.25	76.20
6-Nov-09	-7.24	0.26	79.25
8-Feb-10	-7.24	0.26	79.25
5-Apr-10	-7.25	0.27	82.30
14-Apr-10	-7.25	0.27	82.30
8-Jun-10	-7.24	0.28	85.34

4.4.4.4 Comparison for the Results of Vertical Displacements with Elevation Surveys

The elevation surveys from Table 4.4 are plotted against the results of the numerical analysis as shown in Figure 4.20. It can be seen that time-settlement curves of both data are in agreement.

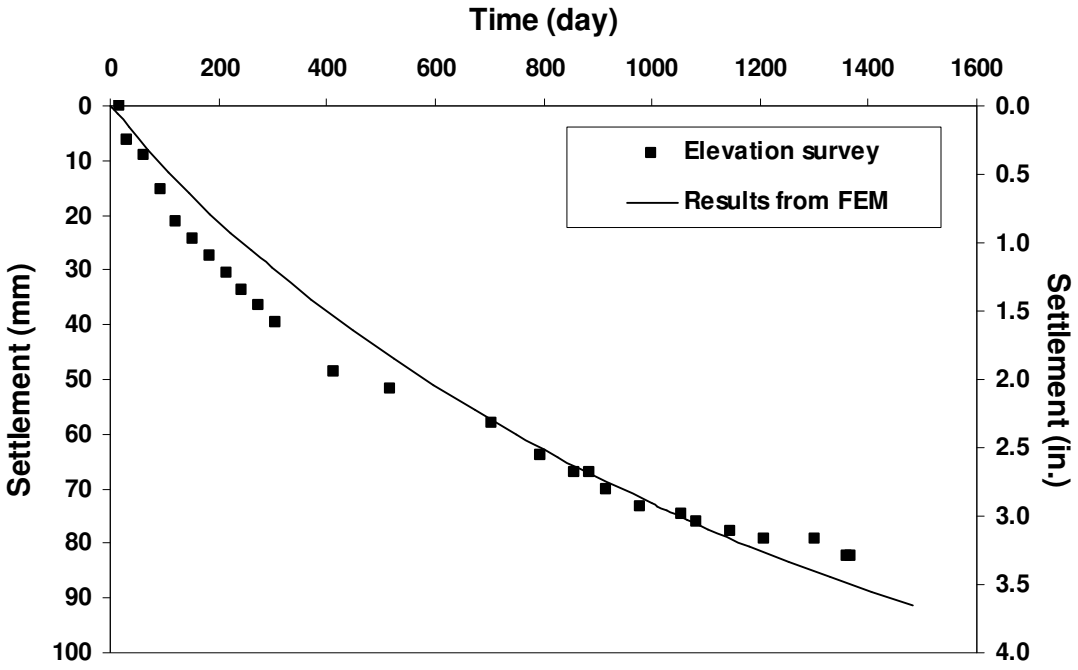


Figure 4.20 Comparison of vertical displacements in a test section between data obtained from elevation surveys and results from numerical analysis

Overall, it can be mentioned that the present FEM analysis using Plaxis has provided reasonable representation of field settlements and trends recorded. This is regarded as a reasonable validation of the numerical modeling task attempted in this thesis research. The same FEM model was used to simulate slopes of different configurations with various soil properties and these results are used to develop design charts. These are discussed in the next section.

4.4.5 Design Charts for Construction of Light Weight Fill Embankments for New Bridges

Design charts were developed for the design of embankments using the light weight fill, Expanded Clay and Shale (ECS). The validated configuration of the embankment from the above studies is considered for the preparation of the design charts. The validated model is then modified with varying heights of embankments, varying thicknesses of the subgrade and also the compression index of the subgrade is varied to increase the scope of application of the design charts. The numerical analysis is performed for a period of 30 years for the development of design charts. From Archeewa (2010), the influence of the embankment slope on the soil displacement in both the vertical and horizontal directions is not significant; hence the influence of the slope is neglected.

Numerical analysis is performed on the validated embankment model by varying the heights of the embankment from 15 ft to 40 ft (5 m to 12 m), while the slope ratio is kept constant at 1:2 (V: H). Figure 4.21 shows point A, which is selected to study in these whole analyses and the embankment model with a height of 20 ft (6.0 m) and slope of 1:2 (V: H). The width of embankment is also varied at its base according to the height of embankment from case to case in order to maintain the slope value at 1 :2.

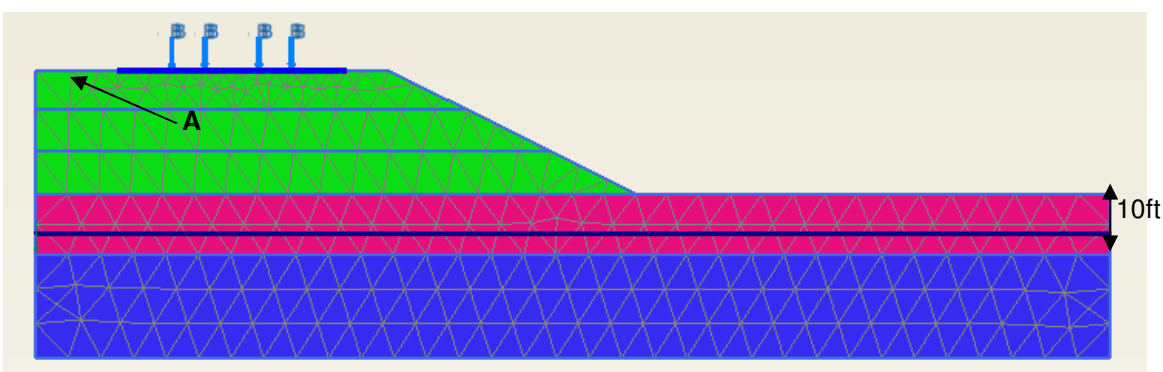
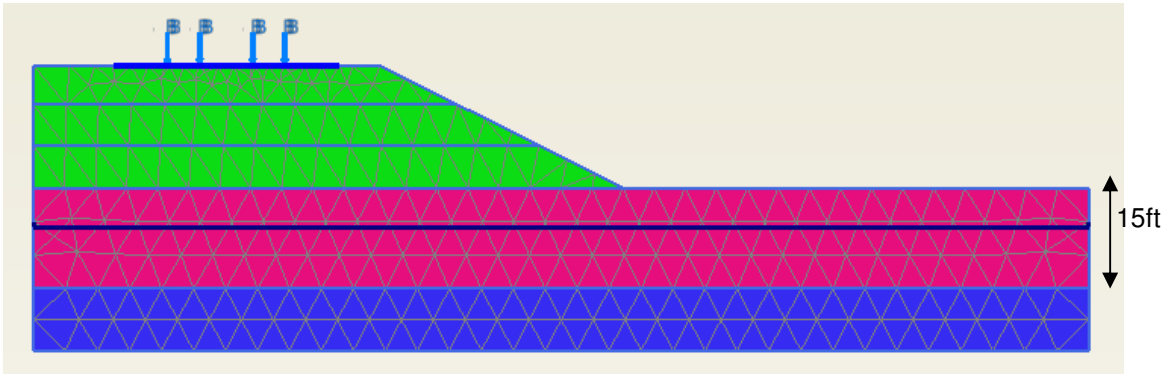
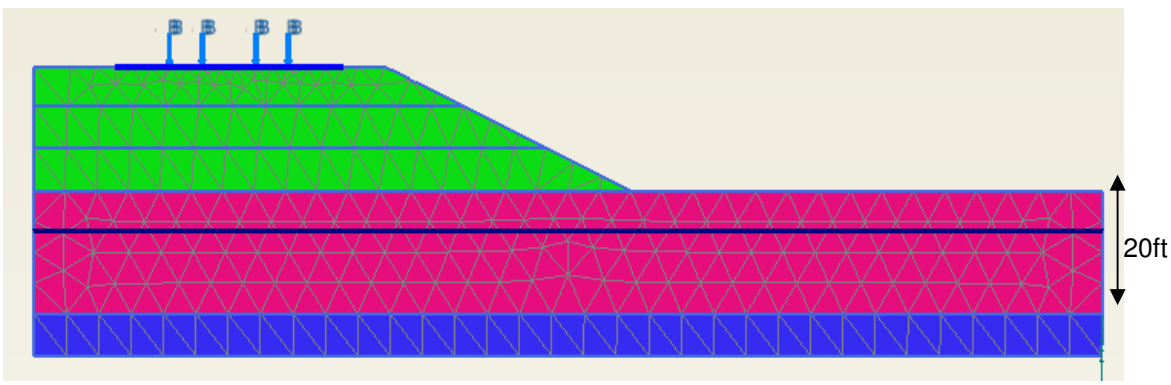


Figure 4.21 Geometry of the test section with a height of 20 ft (6.0 m) and subgrade thickness of 10 ft (5.0 m)

Figure 4.22 (a) and (b) show the embankment model with a height of 20 ft (6.0 m) and slope of 1:2 (V: H) with subgrade thickness 15 ft (5 m) and subgrade thickness 20 ft (6.0 m).



(a)



(b)

Figure 4.22 (a) & (b) Geometry of the test section with a height of 20 ft (6.0 m) and subgrade thicknesses of 15 ft (5 m) and 20 ft (6.0 m) respectively

Design charts were established taking into account different heights of embankment ranging from 15 - 40 ft (5 – 12 m). In addition to varying the thickness of the subgrade, the compression indices of the subgrade were also varied. The compression indices differ from 0.34 to 0.40. The same analysis varying all the parameters mentioned above was performed on the model with local fill also. Considering the results from the numerical analysis of the ECS fill

section and the local fill section and based on the thickness of subgrade, three design charts were established as shown in Figures 4.23, 4.24 and 4.25.

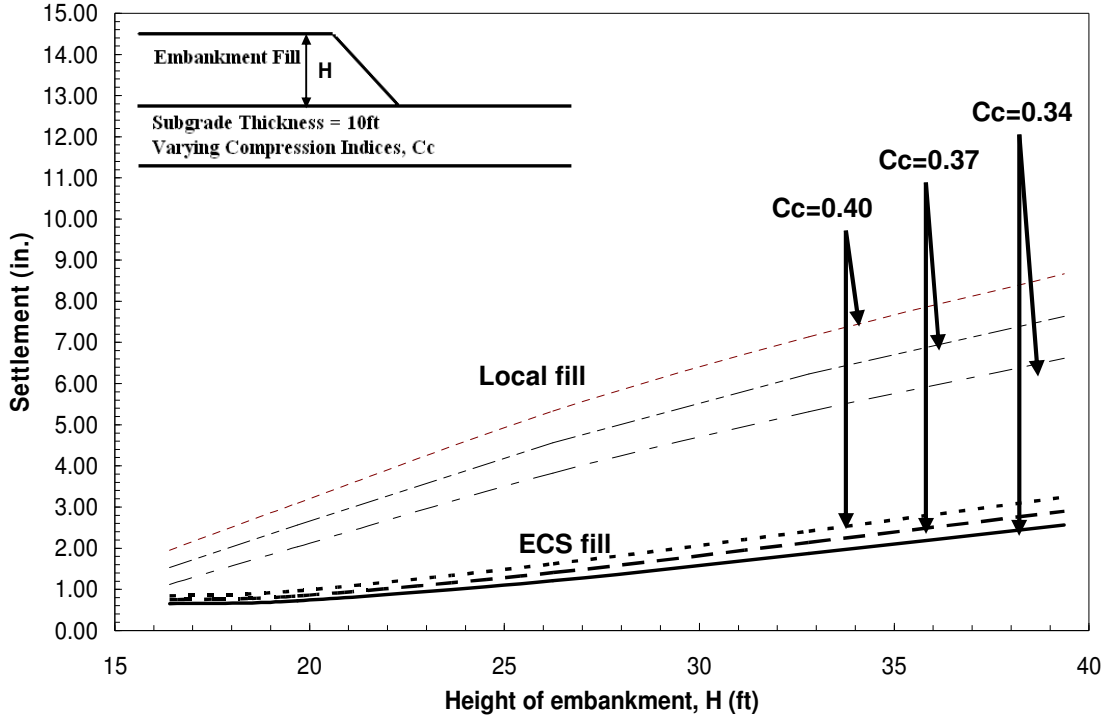


Figure 4.23 Design chart for ECS embankment with subgrade thickness 10 ft

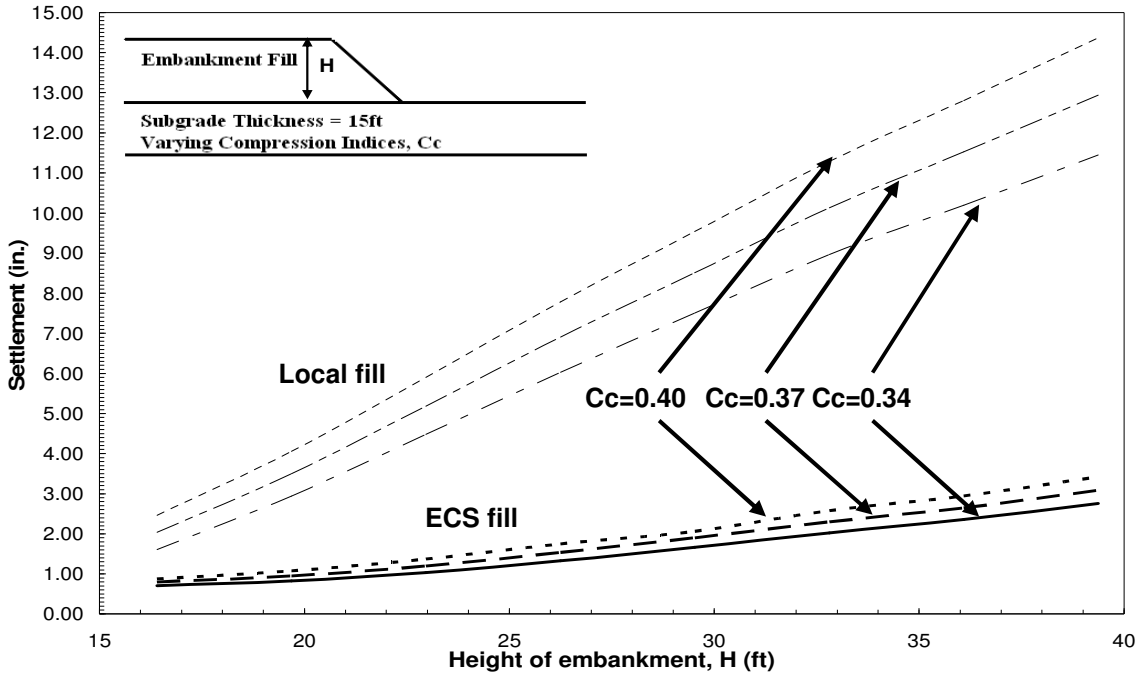


Figure 4.24 Design chart for ECS embankment with subgrade thickness 15 ft

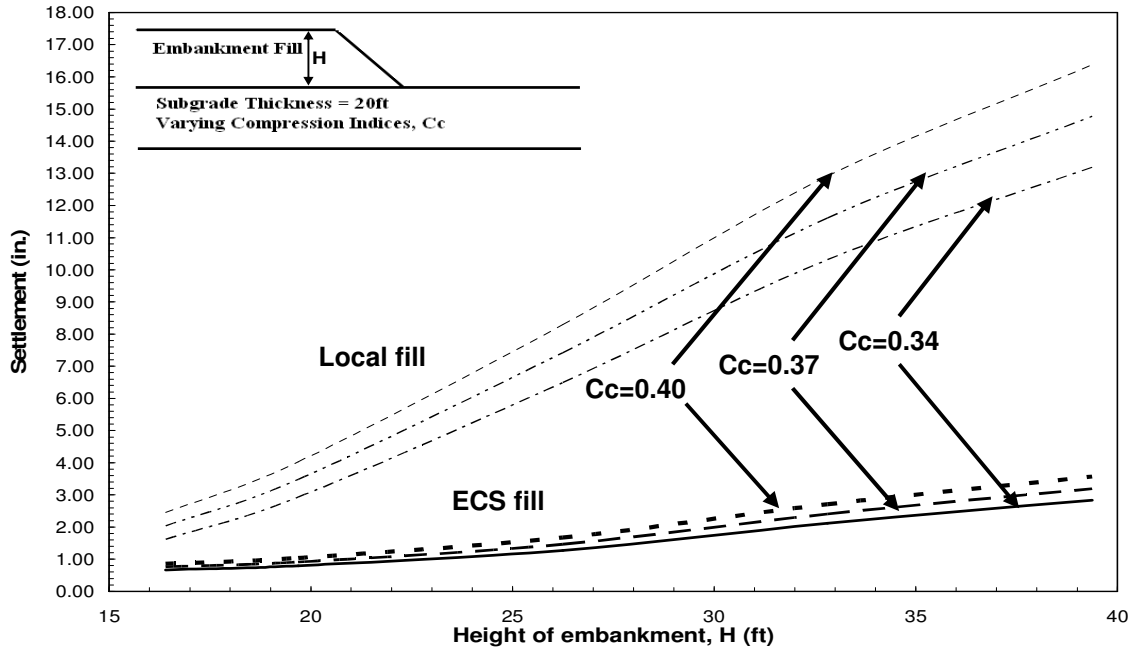


Figure 4.25 Design chart for ECS embankment with subgrade thickness 20 ft

The ECS embankment design charts shown in Figures 4.23 to 4.25, can be interpreted following the design step-by-step procedure;

1. Establish the height of the embankment.
2. Select the appropriate chart from Figure 4.23 to 4.25, based on the thickness of the subgrade.
3. For the given compression index of the subgrade, compare the settlement from the ECS fill embankment section to the local fill embankment section.
4. Select the fill with settlement most nearer to the allowable settlement requirement

4.5 Summary

This chapter presents the details of the numerical analysis used in the establishment of design charts to design embankments using the light weight fill, ECS. The data used as input data for the numerical analysis in the FEM model and the site monitored data used to validate the model is obtained from previous studies by Archeewa (2010). In this study, the analyses are performed on a bridge site on SH360, in Arlington, having a control and a test section to compare the effective reduction in settlement.

To perform the analysis, three tasks were attempted on modeling and validation, modeling with various embankment configurations and establishing design charts for construction of the embankment. For the model validation, the soil parameters from studies by Archeewa (2010) and embankment geometry are used as the input parameters in the model. In this task, after the numerical model had been completely executed, the results from the numerical analyses are compared with the monitored data from the field obtained from Archeewa (2010), to validate the model. The second task is performed to understand how the amount of the settlement occurred in the treated embankment can be affected by various embankment geometries and various subgrade characteristics. The validated numerical model

is used to perform the analysis for the different geometries and subgrade properties with varying embankment heights, varying subgrade thicknesses and varying subgrade compression indices. The last task is the establishment of design charts for the construction of light weight fill embankments using ECS. The results obtained after performing the numerical analysis on both the ECS section and local fill section are used to establish the charts.

From the studies, it can be concluded that using lightweight fill as a backfill material can reduce the settlement occurred in the embankment. Since the gravity load the embankment exerts on the subgrade is a main factor that governs the settlement in the ECS site, the embankment constructed with the lightweight ECS has less amount of settlement than the one constructed with a normal fill; and the higher embankment experiences more settlements than the lower one. However, it should be noted that using the ECS as a backfill material is not a concrete technique to reduce the settlement. Although within 30 years using lightweight fill can reduce the settlement by 30%, this method does not impede the consolidation phenomenon. Nevertheless, the settlement in the embankment, induced by the consolidation, will still exist and remedial techniques are necessary performed in the long-term.

CHAPTER 5

SUMMARY AND CONCLUSIONS

5.1 General

The differential settlements between bridge approach and bridge deck is termed as “the Bump” and this is considered as one of the main problem that affects the performance of the bridge structures. Many state highway agencies in the United States reported this as one of their major maintenance problems as every year these agencies have been spending over $100 million on maintenance and repairs to the bridges and highways damaged by this bump problem. In addition, this bump can cause inconvenience to traveling passengers. Recently, there have been several methods utilized to mitigate this settlement problem such as using driven piles, drilled shafts, Flowable fill, ECS, DSM columns, geosynthetics, and others. In this research, the effectiveness of different mitigation techniques adapted to repair distressed bridge approach slabs and the development of comprehensive design charts to design bridge embankments using light weight embankment fill material using data from a previous research by Archeewa (2010) were discussed.

The objective of this thesis research is to provide appropriate mitigation techniques that can be employed to treat the distressed bridges. Hence four bridge approach slabs with settlement related problems are identified. The location of the bridges, the corrective methods employed, procedure of the method used and the geophysical studies performed on the bridges along with recommendations are discussed. The corrective methods both proposed and recommended include Polyurethane injection, Soil nailing, Flowable fill and the use of light weight Geofoam blocks.

The second objective is the development of design charts for the construction of light weight fill embankments using Expanded Clay and Shale (ECS) aggregate. The data from laboratory studies and data collected in the field were obtained from a previous study by Archeewa (2010). The data from laboratory studies was used as input parameters for the numerical models and the data collected from the field instrumentation was used to validate the model with the results from the FEM. Once validations are done, the models are used for further modeling for hypothetical embankment sections. Both test and control sections are simulated in the FEM model with the embankment geometry and the surcharge loading from traffic and these results were used to develop design charts for the ECS fill embankments.

5.2 Summary and Conclusions

The major objective of the research is to address the effectiveness of each mitigation method in reducing the settlement occurred in the embankment. The following conclusions are obtained for each method considered in the research.

Efficiency of Mitigation Strategies in repairing distressed approach slabs

One of the bridge approach slabs in Waco district where polyurethane injection technique was adopted, was monitored for three years and the settlement of the slab was always within the allowable limits during this monitoring period (less than ¼ in.). However, there was a recent overlay placed on the approach slab along the FM 1947 in June 2011; since the overlay was performed all along the pavement section for several miles, it could be mentioned that the repaired approach slab has performed well without any additional distress.

Some issues with the urethane injection are the need to determine the extent and amount of voids developed underneath the approach slab. GPR techniques are proven to be useful in establishing these void details. This research also recommends a few quality control measures on the urethane material injected and a few field samples are needed to determine the quality of this material used in the field.

In the case of soil nailing, the method was mainly used to enhance the stability of the MSE wall setup, which in turn can control erosion thereby enhancing the stability of the whole bridge including the approach slabs at both ends. Since the soil nailing technique was recently adopted, further monitoring of the bridge is needed to address the use of this technique to control the approach slab settlement that is induced by the MSE wall fill erosion and potential settlement of foundation subgrade.

Two other bridges in Childress and San Antonio districts were also studied for potential implementation of light weight Geofoam blocks and use of Flowable fills. Both methods are considered to have high potential to mitigate settlements. However, these methods are not explored and need to be investigated in future studies.

Efficiency of ECS in reducing settlements in the embankment

The results of vertical soil settlement analysis from the numerical modeling in the ECS aggregate fill test section and the local high plastic soil fill control section are in accordance with the elevation survey data obtained in both the sections. These signify that the models for both sections are accurate enough to estimate soil settlements in the future.

It should be noted that although using the lightweight fill material, like ECS, to construct an embankment can lessen the amount of the settlement to one-third compared with the values predicted in the normal fill embankment in this study, this technique does not impede the consolidation phenomenon occurred in the clay layer. Therefore, remedial works are still necessary to be performed in this type of embankment throughout its long-term service.

The design charts developed can be used in the construction of light weight fill embankments using Expanded Clay and Shale aggregate.

5.3 Limitations and Recommendations

Numerical modeling of soil erosion behavior is a complicate phenomenon and these could not be simulated in the present analysis. Future research should focus on this and also slope stability issues at the steeper slope configurations.

Field studies are needed to evaluate the performance of flowable fills and Geofoams for mitigating the settlements underneath approach slabs.

REFERENCES

[1] Abendroth, R. E., Greimann, L.F. and LaViolette, M. D. (2007). “An Integral Abutment Bridge With Precast Concrete Piles.” *Final Rep., IHRB Project TR-438, CTRE Project No. 99-48*, IOWA Department of Transportation, Center for Transportation Research and Education Report, Ames, Iowa.

[2] Aboshi, H. and Suematsu, N. (1985), “Sand Compaction Pile Method: State-of-the-Art Paper.” *Proc. of 3rd International Geotechnical seminar on Soil Improvement Methods*, Nanyang Technological University, Singapore.

[3] Abu al-Eis, K. and LaBarca, I. K. (2007). “Evaluation of the URETEK Method of Pavement Lifting.” *Rep. No. WI-02-07*, Wisconsin Department of Transprotation, Division of Transportation Systems Development, Madison, Wisconsin.

[4] Abu-Hejleh, N., Hanneman, D., White, D. J. and Ksouri, I. (2006). “Flowfill and MSE Bridge Approaches: Performance, Cost and Recommendations for Improvements.” *Report No. CDOT-DTD-R-2006-2.* Colorado Department of Transportation, Denver

[5] Abu-Hejleh, N., Wang, T. and Zornberg, J.G. (2000). “Performance of Geosynthetic-Reinforced Walls Supporting Bridge and Approaching Roadway Structures.” *ASCE Geotechnical Special Publication No. 103*, Advances in Transportation and Geoenvironmental Systems Using Geosynthetics (2000) 218–243.

[6] Abu-Hejleh, N., Zornberg, J.G., Wang, T. and Watcharamonthein, J. (2003). “Design Assessment of Founders-Meadows GRS Abutment Structure.” *Proc., 82nd Annual TRB Meeting.*

[7] Albajar, L., Gascón, C., Hernando, A. and Pacheco, J. (2005). “Transiciones de Obra de Paso-Terraplén. Aproximación al Estado del Arte y Experiencias Españolas.” Asociación Técnica de Carreteras, Ministerio de Fomento.

[8] American Association of State Highway and Transportation Official AASHTO. (2004). *LRFD Bridge Design Specification, 3rd edition, with 2006 interim revisions,* Washington D.C.

[9] Amini, F. (2004). “Dynamic cone penetrometer in quality control of compaction, state-of-the-art report.” *Proc., Geotrans 2004 – Geotechnical Engineering for Transportation Projects*, Geotechnica; Special Publications No. 126, Yegian and Kavazanjan, eds., ASCE, Los Angeles, Ca., 1023-1031.

[10] Ampadu, S. and Arthur, T. (2006). “The dynamic cone penetrometer in compaction verification on a model road pavement.” *Geotech Test. J.*, ASTM, 29(1), 70-79.

[11] Archeewa, E. (2010). “ Comprehensive studies on deep soil mixing and light weight aggregates applications to mitigate approach slab” December 2010

[12] Arasteh, M. (2007). “Innovations in compaction control and testing.” Presented at 2007 Construction Conference. Bismarck, North Dakota.

[13] Ardani, A. (1987) “Bridge Approach Settlement.” *Report No. CDOT-DTP-R-87-06*, Colorado Department of Highways, Denver, Co.

[14] Arockiasamy, M., Arockiasamy, Butrieng, N. and Sivakumar, M. (2004). “State-of-the-Art of Integral Abutment Bridges: Design and Practice.” *Journal of Bridge Engineering,* ASCE, September/October, 2004, 497-506.

[15] Arsoy, S., Barker, R. M. and Duncan, J. M. (1999). “The Behavior of Integral Abutment Bridges.” *Rep. No. VTRC 00-CR3*, Virginia Transportation Research Council, Charlottesville, VA.

[16] Arsoy, S., Duncan, J. M. and Barker, R.M. (2002). Performance of Piles Supporting Integral Bridges. *Transportation Research Record 1808*, Washington, D.C.

[17] Atkinson, M.S. and Eldred, P. (1981). “Consolidation of soil using vertical drains” *Geotechnique*, 31(1), 33-43.

[18] Bakeer, M., Shutt, M., Zhong, J., Das, S. and Morvant, M. (2005). Performance of Pile-Supported Bridge Approach Slabs.” *Journal of Bridge Engineering*, ASCE.

[19] Barksdale, R.D., and Bachus, R.C. (1983) “Design and Construction of Stone Columns, Vol. 1” *Rep. No. FHWA/RD-83/026*, Federal Highway Administration, Washington, D.C.

[20] Barron, R. A. (1948) “Consolidation of Fine-Grained Soils by Drain Wells”, *Transactions of ASCE No. 2346*, pp. 718-754.

[21] Barron, R. F., Wright, J., Kramer, C., Andrew, W. H, Fung, H. and Liu, C. (2006), “Cement Deep Soil Mixing Remediation of Sunset North Basin Dam” *Dam Safety 2006*, Association of Dam Safety Officials.

[22] Bathe, K.-J. (2003). Finite Element Procedures, Prentice-Hall of India, New Delhi

[23] Baumann, V. and Bauer, G.E.A. (1974). “The Performance of Foundations on Various Soils Stabilized by the Vibro-Compaction Method.” *Canadian Geotechnical Journal*, 509-530.

[24] Bergado, D.T, and Patawaran, M.A.B. (2000). “Recent developments of ground improvement with pvd on soft Bangkok clay.” *Proc. Intl. seminar on Geotechnics in Kochi 2000*, Kochi, Japan, October, 2000.

[25] Bergado, D.T., Anderson, T.R, Miura, N, and Balasubramaniam, A.S. (1996). “Soft Ground Improvement in lowland and other environments.” ASCE, New York.

[26] Bergado, D.T., Miura, N., gh, N. And Panichayatum, B. (1988). “Improvement of Soft Bangkok Clay using Vertical Band Drains Based on Full Scale Test.” *Proc. of the International Conference on Engineering Problems of Regional Soils*, Beijing, China, 379-384.

[27] Bergado, D.T., Singh, N., Sim, S.H., Panichayatum, B., Sampaco, C.L. and Balasubramaniam, A.S. (1990) “Improvement of Soft Bangkok Clay Using Vertical Drains Compacted with Granular Piles.” *Geotextiles and Geomembranes*, 9, 203-231.

[28] Bowders, J., Loehr, E., Luna, R. and Petry, T.M. (2002). “Determination and Prioritization of MoDOT Geotechnical Related Problems with Emphasis on Effectiveness of Designs for Bridge Approach Slabs and Pavement Edge Drains.” *Project Rep. No. R199-029*, Missouri Department of Transportation, Jefferson.

[29] Bowles, J. E. (1988). *Foundation Analysis and Design*, McGraw-Hill, New York.

[30] Bozozuk, M. (1978) “Bridge Foundations Move.” *Transportation Research Record 678*: Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts, Transportation Research Board, National Research Council, Washington, D.C. 17–21.

[31] Brewer, W.E. (1992). “The Design and Construction of Small Span Bridges and Culvert Using Controlled Low Strength Material (CLSM).” *FHWA/OH-93/014*, Ohio Department of Transportation, Columbus, 129.

[32] Brewer, W. B., Hayes, C. J., and Sawyer, S. (1994). “URETEK Construction Report,” *Construction Report*, Report Number OK 94(03), Oklahoma Department of Transportation.

[33] Briaud, J. L., James, R. W., and Hoffman, S. B. (1997) NCHRP synthesis 234: Settlement of Bridge Approaches (the bump at the end of the bridge), Transportation Research Board, National Research Council, Washington, D.C. pp.75.

[34] Bridge Design Manual. Texas Department of Transportation, December 2001.

[35] Brown, D.A., Steve, D.D., Thompson, W.R., and Lazarte, C. A. (2007) "Design and Construction of Continuous Flight Auger (CFA) Piles", Geotechnical Engineering Circular No. 8, Federal Highway Administration, Washington, D. C.

[36] Bruce, D. (2001). "An Introduction to the Deep Mixing Methods as Used in Geotechnical Applications. Volume III. The Verification and Properties of Treated Ground." Report No. FHWA-RD-99-167, US Department of Transportation, Federal Highway Administration, 2001.

[37] Bugrov, A.K. (1975). "The finite element method in consolidation analyses of water-saturated soils", Power Technology and Engineering (Formerly Hydrotechnical Construction), Vol. 9, No. 7, 661-668.

[38] Burde, H.J. (2004). "FE in Geotechnical Engineering", Stichting Postcademish Onderwijs Civile Techniek en Bouwtechniek, Noordwijkerhout, The Netherlands, January, 2004.

[39] Burdette, E.G., S.C. Howard, J.B. Tidwell, E.P. Wasserman, E.E. Ingram, J.H. Deatherage, and D.W. Goodpasture. 2004. Lateral Load Tests on Prestressed Concrete Piles Supporting Integral Abutments. PCI Journal, 49(5), 70–77.

[40] Burke, M.P. (1987). “Bridge Approach Pavements, Integral Bridges, and Cycle-Control Joints.” *Transportation Research Record 1113*, TRB, National Research Council, Washington D.C., 54-65.

[41] Burke, M.P. (1993). Integral bridges: attributes and limitations. Transportation Research Record 1393 p. 1-8. Transportation Research Board, 75th Annual Meeting.

[42] Burke, G. (2001) “Current Methods of Sampling and Testing of Soil Cement and Their Limitations.” Proceedings of International Workshop on Deep Mixing Technology, Vol. 2, National Deep Mixing Program, Oakland, CA. 2001.

[43] Bush, D.I., Jenner, C.G., and Bassett, R.H., 1990. The design and construction of geocell foundation mattress supporting embankments over soft ground. Geotextiles and Geomembranes, 9: 83-98.

[44] Buss, W.E. (1989). “Iowa Flowable Mortar Saves Bridges and Culverts.” Transportation Research Record 1234, TRB, National Research Council, Washington, DC, pp. 30-34.

[45] Byle, M. J. (1997). "Limited Mobility Displacement Grouting - When "Compaction Grout" is Not Compaction Grout." Grouting: Compaction, Remediation, Testing, Proceedings of GeoLogan GeoInstitute Conference, Geotechnical Special Publication No. 66, Logan, Utah, ASCE, p. 32-43.

[46] Byle, M. J. (2000). "An Approach to the Design of LMD Grouting." *Geotechnical Special Publication 104*, Advances in Grouting and Ground Modification, Proceedings of Sessions of Geo-Denver 2000, Wallace Baker Memorial Symposium, ASCE, Denver, Colorado, 94-110.

[47] Cai, C. S., Voyiadjis, G. Z. and Xiaomin, S. (2005). “Determination of Interaction between Bridges Concrete Approach Slab and Embankment Settlement.” Department of Civil Engineering, Louisiana State University, Baton Rouge, La.

[48] Chini, S. A., Wolde-Tinsae, A. M., and Aggour, M. S. (1992). “Drainage and Backfill Provisions for Approaches to Bridges.” University of Maryland, College Park.

[49] Cooper, M.R. and Rose, A.N. (1999). “Stone column support for an embankment on deep alluvial soils.” *Proc of the Institution of Civil Engineer*s, Geotechnical Engineering, 137(1),15-25.

[50] Cotton, D. M., Kilian, A. P., and Allen, T. (1987). “Westbound Embankment Preload on Rainier Avenue, Seattle, Washington.” *Transportation Research Record No. 1119*, Transportation Research Board, Washington D.C., 61-75.

[51] Cowland. J.W., and Wong, S.C.K. (1993). “Performance of a road embankment on soft clay supported on a geocell mattress foundation.” *Geotextiles and Geomembranes*, 12, 687-705.

[52] Das, S. C., Bakeer, R., Zhong, J., and Schutt, M. (1990). “Assessment of mitigation embankment settlement with pile supported approach slabs.” Louisiana Transportation and Research Center, Baton Rouge, La.

[53] DeMerchant, M., R. and Valsangkar, A., J. (2002). “Plate load tests on geogrid-reinforced expanded shale lightweight aggregate.” *Geotextiles and Geomembranes No. 20*, 173–190

[54] DiMillio, A.F. (1982). “Performance of Highway Bridge Abutments Supported by Spread Footings on Compacted Fill.” *Final Rep. No. FHWA/RD-81/184*, Federal Highway Administration, Washington, D.C.

[55] Du, L. (2008). Personal communication, Texas Department of Transportation.

[56] Du, L., Arellano, M., K. J. Folliard, Nazarian, S. and Trejp D. (2006). “Rapid-Setting CLSM for Bridge Approach Repair: A Case Study”. ACI Material Journal/ September-October 2006, pp. 312-318.

[57] Dumas, C., Munsukhani, S., Porbaha, A., Short, D.R., Cannon, R.R., McLain, W.K., Putcha, S., Macnub, A., Lwin, M.M., Pelnik III, T.W., Brown, D.A. and Christopher, R.B. (2003). “Innovative Technology for Accelerated Construction of Bridge and Embankment Foundations in Europe”, *FHWA, Rep. No. PL-03-014.*

[58] Dunn, K. H. Anderson, G.H. Rodes, T.H., and Zieher, J.J. (1983). “ Performance Evaluation of Bridge Approaches.” Wisconsin Department of Transportation.

[59] Dupont, B. and Allen, D. (2002). “Movements and Settlements of Highway Bridge Approches.” *Rep. No. KTC-02-18/SPR-220-00-1F*, Kentucky Transportation Center Report, Lexington, KY.

[60] EBA Engineering, Inc. (1992). *Final Report: Summary results obtained from the auger cast piles investigation. Study DTFH 692-Q-00062*. FHWA, U.S. Department of Transportation.

[61] Edgar, T. V., Puckett, J. A., and D’Spain, R. B. (1989). “Effect of Geotextile on Lateral Pressure and Deformation in Highway Embankments.” *Geotextiles and Geomembranes*, 8(4), 275-306.

[62] Elias, V., Christopher, B. R., and Berg, R.R. (2001). “Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines.” FHWA NHI-00- 043.

[63] Expanded Shale, Clay and Slate (ESCS) (2004). “Light weight aggregate for geotechnical applications.” Information Sheet 6001 – www.escci.org.

[64] Federal Highway Administration (2000). “Priority, Market-Ready Technologies and Innovations”, *Rep. No. FHWA-HRT-04-053*.

[65] Fox, N.S. and Cowell, M.J. (1998). *Geopier Foundation and Soil Reinforcement Manual*, Geopier Foundation Company, Inc., Scottsadale, AZ.

[66] Gabr, M., Hopkins, K., Coonse, J, and Hearne, T. (2000). “DCP criteria for performance evaluation of pavement layers.” *J. Perf. Of Constr. Facil.*, ASCE, 14(4), 141-148.

[67] Gallivan, L. (2008). “An Intelligent Compaction Era has Arrived.” *Rep. No. FHWA-HRT-08-012*, Focus, USDOT.

[68] Girton, D. D., Hawkinson, T.R. and Greimann, L.F. (1991). “Validation of design recommendations for integral-abutment piles.” *Journal of Structural Engineering*, ASCE, 117(7), 2117-2134.

[69] Goughnour, R. R. and Bayuk, A.A. (1979). “A Field Study of Long-term Settlement of Loads Supported by Stone Column in Soft Ground.” *Proc of International Conference on Soil Reinforcement: Reinforced Earth and Other Techniques*, Vol. 1, Paris, 279-286.

[70] Greimann, L. F. and Wolde-Tinsae, A. M. (1988). “Design model for piles in jointless bridges.” *Journal of Structural Engineering*, ASCE, 114(6),1354-1371.

[71] Greimann, L. F., Abendroth, R.E., Johnson, D.E. and Ebner, P.B. (1987). “Pile Design and Tests for Integral Abutment Bridges.” *Final Rep. Iowa DOT Project HR 273, ERI Project 1780, ISU-ERI-Ames-88060*, Ames, Iowa.

[72] Greimann, L. F., Yang, P-S. and Wolde-Tinsae, A. M. (1983). “Skewed Bridges with Integral Abutments.” *Transportation Research Record 903*: Bridges and Culverts, Transportation Research Board, National Research Council, Washington, D.C., 64-72.

[73] Greimann, L. F., Yang, P-S. and Wolde-Tinsae, A. M. (1986). “Nonlinear analysis of integral abutment bridges.” *Journal of Structural Engineering*, ASCE, 112(10), 2263-2280.

[74] Grover, R. A. (1978). “Movement of Bridge Abutments and Settlements of Approach Pavements in Ohio.” *Transportation Research Record 678*, Washington, D.C.

[75] Ha, H.,Seo, J., and Briaud, J-L. (2002). “ Investigation of Settlement at Bridge Approach Slab Expansion Joint: Survey and Site Investigations.” *Rep. No. FHWA/TX-03/4147-1,* Texas Transportation Institute, Texas A&M University, College Station, TX.

[76] Hansbo, S. (1979) “Consolidation of Clay by Bandshaped Prefabricated Drains.” *Ground Engineering*, 12(5), 16-25.

[77] Hansbo, S. (1997) “Aspects of Vertical Drain Design: Darcian or Non-Darcian Flow.” *Geotechnique*, 47(5), 983-992.

[78] Hansbo, S. (1997) “Practical Aspects of Vertical Drain Design.” *Proceedings of 14th International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, Hamburg.

[79] Hansbo, S. (2001) “Consolidation Equation Valid for both Darcian or Non-Darcian Flow”, *Geotechnique*, 51(1), 51-54.

[80] Hartlen, J. (1985). “Pressure Berms, Soil Replacement and Lightweight Fills.” *Soil Improvement Methods, Proceedings of the Third International Geotechnical Seminar*, Nangyang Technological Institute, Singapore, 101-111.

[81] Hausmann, M. R. (1990). “Engineering Principles of Ground Modification”, McGraw-Hill Publishing Company, New York.

[82] Holm, T., A., and Valsangkar, A., J. (1993). “Lightweight aggregate soil mechanics: properties and applications.” Transportation Research Record, No. 1422 , 7-13.

[83] Holm, T. A., and Ooi, O. S. (2003). “Moisture dynamics in lightweight aggregate and concrete.” 6th International Conference on the Durability of Concrete, Thessaloniki, Greece.

[84] Holtz, R.D. and Kovacs, W.D. (1981). “An Introduction to Geotechnical Engineering.” Prentice Hall Hall Inc., Englewoods Cliffs, New Jersey, 733.

[85] Hopkins, T.C. (1969). “Settlement of Highway Bridge Approaches and Embankment Foundations.” *Rep. No. KYHPR-64-17; HPR-1(4)*, Kentucky Transportation Center, Lexington, Kentucky.

[86] Hopkins, T. C. (1973). “Settlement of Highway Bridge Approaches and Embankment Foundations.” *Rep. No. KYHPR-64-17; HPR-1(8)*, Kentucky Transportation Center, Lexington, Kentucky, 40.

[87] Hopkins, T.C. (1985). “Long-Term Movements of Highway Bridge Approach Embankments and Pavements”. University of Kentucky, Transportation Research Program.

[88] Hopkins, T.C. and Deen, R.C. (1970). “The Bump at the End of the Bridge.” *Highway Research Record No. 302*, Highway Research Board, National Research Council, Washington, D.C., 72-75.

[89] Hoppe, E. J. and Gomez, J. P. (1996). “Field study of an integral backwall bridge.” *Rep. No. VTRC 97-R7*, Virginia Transportation Research Council, Charlottesville, VA.

[90] Hoppe, E.J. (1999). “Guidelines for the Use, Design, and Construction of Bridge Approach Slabs.” *Rep. No. VTRC 00-R4*, Virginia Transportation Research Council, Charlottesville, VA.

[91] Horvath, J. (2000). “Integral Abutment Bridges: Problem and Innovative Solutions Using EPS Geofoam and other Geosynthetics.” Research *Report No. CE/GE-00-2*, Manhattan College, Bronx, New York.

[92] Horvath, J. S. (1991). “Using Geosynthetics to Reduce Surcharge-Induced Stresses on Rigid Earth Retaining Structures.” *Transportation Research Record 1330*, 47-53.

[93] Horvath, J. S. (2005). “Integral-Abutment Bridges: Geotechnical Problems and Solutions Using Geosynthetics and Ground Improvement.” *IAJB 2005 - The 2005 FHWA Conference on Integral Abutment and Jointless Bridges*. 16-18 March, Baltimore, Maryland, USA.

[94] Hsi, J. and Martin, J. (2005). “Soft Ground Treatment and Performance, Yelgun to Chinderah Freeway, New South Wales, Australia.” *In Ground Improvement-Case Histories*, Elsevier Geo-Engineering Book Series, Volume 3, 563-599.

[95] Hsi, J. P. (2007). “Managing Difficult Ground-Case Studies.” *Proceedings of First Sri Lankan Geotechnical Society International Conference on soil and Rock Engineering*, Colombo.

[96] Hsi, J. P. (2008). “Bridge Approach Embankments Supported on Concrete Injected Columns.” *Proceedings of The Challenge of Sustainability in the Geoenvironment*, ASCE, Geocongress 08, New Orleans, Louisiana.

[97] HVJ Associates (2003). Report of Geotechnical Investigation Reconstruction and Widening of US-90A at SH-6, Segment 3, Houston, Texas.

[98] Indraratna, B., Balasubramaniam, A. S. and Ratnayake, P. (1994). “Performance of Embankment Stabilized with Vertical drains on Soft Clay.” *Journal of Geotechnical Engineering*, ASCE, 120(2), 257-273.

[99] James, R.W., Zhang, H. and Zollinger, D.G. (1991). “Observations of severe abutment backwall damage.” *Transportation Research Record 1319*, Transportation Research Board, 55-61.

[100] Jamiolkowski, M., Lancellota, R. and Wolski, W. (1983). “Precompression and speeding up consolidation.” *Proceedings, Eight European Conference on Soil Mechanics and Foundation Engineering*, Vol 3, Helsinki.

[101] Japan Railway (JR) Technical Research Institute (1998). “Manual on Design and Construction of Geosynthetic-Reinforced Soil Retaining Wall.”

[102] Jayawickrama, P., Nash, P., Leaverton, M. and Mishra, D. (2005). “Water Intrusion in Base/Subgrade Materials at Bridge Ends.” *TxDOT Report, FHWA/TX-06/0-5096-1*, Texas Tech University, Lubbock, Texas.

[103] Jenner, C.G., Basset, R.H., and Bush, D.I. (1988). “The use of slip line fields to assess the improvement in bearing capacity of soft ground given by cellular foundation mattress installed at the base of an embankment.” *In Proceedings of the International Geotechnical Symposium on Theory and Practice of Earth Reinforcement*, Balkema, Rotterdam, 209-214.

[104] Kamel, M.R., Benak, J.V., Tadros, M.K. and Jamshidi, M. (1996). “Prestressed Concrete Piles in Jointless Bridges.” *PCI Journal*, 41(2), 56–67.

[105] Kramer, S. L. and Sajer, P. (1991) “Bridge Approach Slab Effectiveness.” *Final Report*, Washington State Department of Transportation, Olympia, Washington.

[106] Krishnaswamy, N.R., Rajagopal, K., and Madhavi Latha, G. (2000). “Model studies on geocell supported embankments constructed over soft clay foundation.” *Geotechnical Testing Journal*, ASTM, 23: 45-54.

[107] Kristiansen. H. and Davies, M. (2004). “Ground improvement using rapid impact compaction.” *Proceedings of 13th World Conference on Earthquake Engineering*, Vancouver, B.C, pp 1-10.

[108] Kunin, J. and Alampalli, S. (2000). “Integral Abutment Bridges: Current Practice in United States and Canada.” *ASCE Journal of Performance of Constructed Facilities*, 14(3), 104-111.

[109] Laguros, J. G., Zaman, M. M., and Mahmood, I. U. (1990). “Evaluation of Causes of Excessive Settlements of Pavements Behind Bridge Abutments and their Remedies; Phase II. (Executive Summary).” *Rep. No. FHWA/OK 89 (07)*, Oklahoma Department of Transportation.

[110] Lawton, E.C., and Fox, N.S. (1994). “Settlement of Structures Supported on Marginal or Inadequate Soils Stiffened with Short Aggregate Piers.” *ASCE GSP No. 40,* Vertical and Horizontal Deformations of Foundations and Embankments, ASCE, Vol. 2, 962-974.

[111] Lawton, E. C., Fox, N. S. and Handy, R. L. (1994). “ Control of Settlement and Uplift of Structures Using Short Aggregate Piers.” Proceedings, IN-SITU Deep Soil Improvements.

[112] Lawton, E., and Merry, S. (2000). “Performance of Geopier Supported Foundations During Simulated Seismic Tests on Northbound Interstate 15 Bridge Over South Temple, Salk Lake City, Utah.” Final Reprt No. UUCVEEN 00-03. University of Utah.

[113] Lawver, A., French, C., and Shield, C. K. (2000). “Field Performance of an Integral Abutment Bridge.” *Transportation Research Record*: Journal of the Transportation Research Board, 1740, 108-111.

[114] Lenke, L. R. (2006). “Settlement Issues-Bridge Approach Slabs.” *Rep. No. NM04MNT-02*, New Mexico Department of Transportation.

[115] Lien, B. H. and Fox, N. S. (2001) “Case Histories of Geopier® Soil Reinforcement for Transportation Applications.” Proc. of Asian Institute of Technology Conference, Bangkok, Thailand.

[116] Lin, Q.L., Wong, I.H. (1999). “Use of Deep Cement Mixing to Reduce Settlements at Bridge Approaches.” Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 125(4),309.

[117] Liu, H.L., Ng, C.W.W., and Fei, K. (2007). “Performance of a Geogrid-Reinforced and Pile-Supported Highway Embankment over Soft Clay: Case Study.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 133(12),1483-1493.

[118] Long, J.H., Olson, S.M. and Stark, T.D. (1998). “Differential Movement at Embankment/Bridge Structure Interface in Illinois.” *Transportation Research Record No. 1633*, Transportation Research Board, Washington, D.C., 53-60.

[119] Lukas, R.G. (1986) “Dynamic Compaction for Highway Construction, Vol. 1, Design and construction Guidelines.” *Rep. No. FHWA/RD-86/133* , Federal Highway Administration, National Technical Information Service, Washington, D.C.

[120] Lukas, R.G. (1995) “Dynamic Compaction, Geotechnical Engineering Circular No. 1” *Rep. No. FHWA-SA-95-037*, Federal Highway Administration,Washington, D.C.

[121] Luna R., Jonathan L. R., and Andrew, J. W. (2004). “Evaluation of Bridge Approach Slabs, Performance and Design.” *Rep. No. RDT 04-010*, Department of Civil, Architectural and Environmental Engineering, University of Missouri, Rolla.

[122] Maddison, J. D., Jones, D. B., Bell, A. L., and Jenner, C. G. (1996). “Design and performance of an embankment supported using low strength geogrids and vibro concrete columns.” *Geosynthetics: Applications, design and construction, De Groot, Den Hoedt, and Termaat, eds.*, Balkema, Rotterdam, The Netherlands, 325–332.

[123] Magnan, J. (1994). “Methods to reduce the settlement of embankments on soft clay: A review.” *Proc., Vertical-Horizontal Deformations of Foundations and Embankments*, ASCE, New York, 77–91.

[124] Mahmood, I. U. (1990). “Evaluation of causes of bridge approach settlement and development of settlement prediction models.” PhD Thesis, University of Oklahoma, Norman, Okla.

[125] McKenna, J.M., Eyre, W.A., and Wolstenholme, D. R. (1975). “Performance of an Embankment Supported by Stone Columns in Soft Ground.” *Geotechnique*, 25(1), 51-59.

[126] McVay, M., Armaghani, B. and Casper, R. (1994). “Design and construction of auger-cast piles in Florida.” *Transportation Research Record, 1447*, 10-18.

[127] Mekkawy, M., White, D.J., Souleiman, M.T. and Sritharan, S. (2005). “Simple Design Alternatives to Improve Drainage and Reduce Erosion at Bridge Abutments.” *Proceedings of the 2005 Mid-Continent Transportation Research Symposium*, Ames, Iowa.

[128] Michell, J. K., and Huber, T.R (1985) “Performance of a Stone Column Foundation.” *Journal of Geotechnical Engineering*, ASCE, 111(2), 205-223.

[129] Miller, E. A., and Roykroft, G. A., (2004). “Compaction Grouting Test Program for Liquefaction Control.” *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, 355-361.

[130] Mimura, C.S., Kimura, S.A. (1995). “A light-weight solution.” *Proceedings Geosynthetics '95*, Nashville, Tennessee, Vol. 1, 39-51.

[131] Minks, A.G., Wissmann, K.J., Caskey, J.M., and Pando, M.A. (2001). “Distribution of Stresses and Settlements below Floor Slabs Supported by Rammed Aggregate Piers.” *Proceedings of 54th Canadian Geotechnical Conference*, Calgary, Alberta, Canada, 16-19.

[132] Monley, GJ., and Wu, J. T. (1993). Tensile Reinforced Effects on Bridge Approach Settlement. *Journal of Geotechnical Engineering*, 119(4), 749-763.

[133] Moore, W. (2006). “Intelligent compaction: Outsmarting Soil and Asphalt.” www.constructionequipment.com/article/CA6321917.html.

[134] Munoz, A. and Mattox, M. (1977) “Vibroreplacement and Reinforced Earth Unite to Strengthen a Weak Foundation.” Civil Engineering, ASCE, 58-62.

[135] Narsavage, P. (2007). “Downdrag in Foundation Design.” Ohio Department of Transportation. www.odotnet.net/geotechnical/Web_Links/Docs/2007_Consult_Workshop/Downdrag.ppt.

[136] Nassif, H. (2002). “Finite Element Modeling of Bridge Approach and Transition Slabs.” *Rep. No. FHWA-NJ-2002-007*, Department of Civil and Environmental Engineering, Center for Advanced Infrastructure & Transportation (CAIT), Rutgers, New Jersey.

[137] Neely, W. J. (1991). “Bearing capacity of auger-cast piles in sand”. *Journal of Geotechnical Engineering*, 117(2), 331-345.

[138] Nicholson D.P. and Jardine R.J. (1982). “Performance of vertical drains at Queenborough bypass.” Vertical Drains, the Institution of Civil Engineers: London, 67-90.

[139] O’Neill, M. W. (1994). “Review of augered pile practice outside the United States”. *Transportation Research Record, 1447*, 3-9.

[140] ODOT. (2005). “Bridge Foundation Design Practices and Procedures.” Bridge Engineering Section, Oregon Department of Transportation, 20.

[141] Opland, W. H., and Barnhart, V. T. (1995). "Evaluation of The URETEK Method for Pavement Undersealing." *Research Rep. No. R-1340*, Michigan Department of Transportation in Cooperation with the U.S. Department of Transportation.

[142] PCI. (2001). "The State of the Art of Precast/Prestressed Integral Bridges." Precast/Prestressed Concrete Institute, Chicago, Illinois.

[143] Pierce, C. E., Baus, R. L., Harries, K. A. and Yang, W. (2001) "Investigation into Improvement of Bridge Approaches in South Carolina", *Summary Report, Rep. No. FHWA-SC-01-02*, South Carolina Department of Transportation.

[144] Porbaha, A. (1998). "State of the Art in Deep Mixing Technology, Part I: Basic Concepts and Overview of Technology." *Ground Improvement*, 2(2), 81-92.

[145] Porbaha, A. (2000). "State of the Art in Deep Mixing Technology: Design Considerations." *Ground Improvement*, 4(3), 111-125.

[146] Porbaha, A. and Roblee, C. (2001). "Challenges for Implementation of Deep Mixing in the USA." *Proceedings of International Workshop on Deep Mixing Technology*, National Deep Mixing Program, Oakland, CA., 2 volumes.

[147] Rahman, F., Hossain, M. and Romanoschi, S. (2007). "Intelligent Compaction Control of Highway Embankments Soil." *86th Annual Meeting of the Transportation Research Board*, Washington, D.C., 2007.

[148] Rathmayer, H. (1996). “Deep Mixing Methods for Soft Subsoil Improvement in the Nordic Countries.” *Proceedings of IS-Tokyo ’96*, The 2nd International Conference on Ground Improvement Geosystems, 14-17 May 1996, Tokyo, 869-878.

[149] RECON (2007). DSM Work Plan Report for Project: STP 2007 (165) MM IH 30 Tarrant County, Houston Texas

[150] Report on Texas Bridges (2006). Prepared by the Bridge Division of the Texas Department of Transportation, Austin, Texas.

[151] Rixner, J.J., Kraemer, S.R., and Smith, A.D. (1986). “Prefabricated Vertical Drains, Vol. 1.” *Rep. No. FHWA-RD-86/168*, Federal Highway Administration,Washington, D.C.

[152] Rowe, R.K., Gnanendran, C.T., Landva, A.O. and Valsangkar, A.J. (1995). “Construction and performance of a full-scale geotextile reinforced test embankment.” *Canadian Geotechnical Journal*, 32(3), 512–534.

[153] Saride, S, Sirigiripet, S.K, Puppala, A. J, Williammee, R. (2008). “Performance of Expanded Clay Shale (ECS) as an Embankment Backfill.” In the Proceedings of International Conference on ‘The Challenges of Sustainability in the Geoenvironment’, American Society of Civil Engineers, Geocongress- 2008. March 9-12, New Orleans, Louisiana, USA.

[154] Schaefer, V.R, Koch, J.C. (1992). “Void Development under Bridge Approaches,” *Rep. No. SD90-03*, South Dakota Department of Transportation.

[155] Seo, J. (2003). “The Bump at the End of the Bridge: An Investigation.” Dissertation submitted in partial fulfillment of the requirements for the degree of the Doctor of Philosophy, Texas, A&M University, College Station, Texas.

[156] Seo, J., Ha, H. S., and Briaud, J.L. (2002). “Investigation of Settlement at Bridge Approach Slab Expansion Joint: Numerical Simulation and Model Tests.” *Rep. No. FHWA/TX-03/4147-2*, Texas Transportation Institute, Texas A&M University, College Station, Texas.

[157] Serridge, C.J. and Synac, O. (2007). “Ground Improvement Solutions for Motorway Widening Schemes and New Highway Embankment Construction Over Soft Ground.” *Ground Improvement Journal*, 11(4), 219-228.

[158] Shen, S.-L., Hong, Z.-S., and Xu, Y.-S. (2007). “Reducing Differential Settlements of Approach Embankments.” *Proc. of Institute of Civil Engineers*, Geotechnical Engineering, Issue GE4, 215-226.

[159] Siemeier, J.A., Young, D. and Beberg, D. (2000). “ Comparison of the dynamic cone penetrometer with other tests during subgrade and granular base characterization in Minnesota.” Nondestructive Testing of Pavements and Backcalculation of Moduli: Third Volume, ASTM STP 1375, Tayabji and Lukanen, eds., ASTM, West Conshohocken, Pa., 175-188.

[160] Sluz, A., Sussmann, T. R., and Samavedam, G. (2003). “Railroad Embankment Stabilization Demonstration for High-Speed Rail Corridors.” *Grouting and Ground Treatment* (GSP No. 120). 3rd International Specialty Conference on Grouting and Ground Treatment. February 10–12, New Orleans, Louisiana, USA.

[161] Smadi, O. (2001). “The strength of flowable mortar.” http://www.ctre.iastate.edu/PUBS/tech_news/2001/julaug/flowable_mortar.pdf

[162] Snethen, D. R. and Benson, J. M. (1998). “Construction of CLSM Approach Embankment to Minimize the Bump at the End of the Bridge.” *The Design and Application of Controlled Low-Strength Materials (Flowable Fill)*, ASTM STP 1331.

[163] Soltesz, S. (2002). “Injected Polyurethane Slab Jacking.” *Final Report*, Rep. No. FHWA-OR-RD-02-19, Oregon Department of Transportation Research Group.

[164] Stark, T. D., Olson, S. M., and Long, J. H. (1995). “Differential Movement at the Embankment/Structure Interface Mitigation and Rehabilitation.” *Rep. No. IABH1, FY 93*, Illinois Department of Transportation, Springfield, Illinois.

[165] Stewart, C. F. (1985). “Highway Structure Approaches.” California Department of Transportation, Sacramento, CA.

[166] Stoll, R.D. and Holm, T.A. (1985). “Expanded shale lightweight fill: geotechnical properties.” *Journal of Geotechnical Engineering*, ASCE 111 (GT8), 1023–1027.

[167] Strauss, J., Dahnke, D. and Nonamaker, F.(2004) " Compaction Grouting to Mitigate Settlement Beneath Approach Fills, California State Route 73 at Laguna Canyon Road", *Geotechnical Engineering for Transportation Project*, ASCE,1876-1883.

[168] Tadros, M.K., and Benak, J.V. (1989). "Bridge Abutment and Approach Slab Settlement (Phase I Final Report), University of Nebraska, Lincoln.

[169] Terzaghi, K. (1943) "Theoretical Soil Mechanics." John Wiley and Sons, New York.

[170] Timmerman, D.H. (1976). "An Evaluation of Bridge Approach Design and Construction Techniques." *Final Report*, Rep. No. OHIODOT-03-77, Ohio Department of Transportation.

[171] Transportation Research Board. (2008). "Intelligent Soil Compaction Systems." *Rep. No. NCHRP 21-09*. Washington, D.C.

[172] Unified Facilities Criteria (UFC). (2004). "Pavement Design For Roads, Streets, Walk, and Open Storage Area." *Rep. No. UFC 3-250-01FA*, US Army Corps of Engineers.

[173] Vipulanandan, C., Kim, M.G., and O'Neill, M.W. (2004). "Axial Performance of Continuous Flight Auger Piles for Bearing", *Project Report No. 7-3940-2*, Texas Department of Transportation, Austin, Texas.

[174] Wahls, H. E. (1990). *NCHRP Synthesis of Highway Practice No. 159: Design and Construction of Bridge Approaches*. Transportation Research Board, National Research Council, Washington, D.C.

[175] Walkinshaw, J. L. (1978). “Survey of Bridge Movements in the Western United States.” *Transportation Research Record 678: Tolerable Movements of Bridge Foundations, Sand Drains, K-Test, Slopes, and Culverts*, Transportation Research Board, National Research Council, Washington, D.C., 6–12.

[176] White, D. J. and Suleiman, M.T. (2004). “Design of Short Aggregate Piers to Support Highway Embankments.” 83rd Annual Meeting Transportation Research Board, Washington, D.C.

[177] White, D., Mohamed, M., Sritharan, S. and Suleiman, M. (2007). “Underlying” Causes for Settlement of Bridge Approach Pavement Systems.” *Journal of Performance of Constructed Facilities*, ASCE, 273-282.

[178] White, D., Sritharan, S., Suleiman, M., Mohamed M. and Sudhar, C. (2005). “Identification of the Best Practices for Design, Construction, and Repair of Bridge Approaches.” *CTRE. Project 02-118*, Iowa State University. Ames, Iowa.

[179] White, D.J., Wissman, K., Barnes, A.G. and Gaul, A.J. (2002). “Embankment Support: A Comparison of Stone Column and Rammed Aggregate Pier Soil Reinforcement.” *Proc. 81st Annual TRB Meeting*, Washington, D. C.

[180] Williammee, R. (2008) Personal communication, Fort Worth District Materials Engineer, Texas Department of Transportation.

[181] Wissmann, K. J. and Fox, N.S. (2000). “Design and analysis of Short Aggregate Piers to reinforce soils.” Preceedings, Geotechnical Colloquium, Technical University Darmstadt, Fermany, March 25, 2000.

[182] Wolde-Tinsae, A.M, Aggour, S.M. and Chini, S.A. (1987). “Structural and Soil Provisions for Approaches to Bridges.” *Interim Report AW087-321-046*, Maryland Department of Transportation.

[183] Won, M. S. and Kim, Y. S. (2007). “Internal Deformation Behavior of Geosynthetic-reinforced Soil Walls.” *Geotextiles and Geomembranes*, 25(1), 10-22.

[184] Wong, H. K. W, and Small, J. C. (1994). “Effect of Orientation of Bridge Slabs on Pavement Deformation.” *Journal of Transportation Engineering*, 120(4), 590-602.

[185] Wu, J.T.H., Lee, K.Z.Z., Helwany, B. S. and Ketchart, K. (2006). “Design and Construction Guidelines for Geosynthetic-Reinforced Soil Bridge Abutments with a Flexible Facing.” *Proc. NCHRP Report 556*. Transportation Research Board, Washington DC.

[186] Wu, J.T.H., Lee, K.Z.Z. and Ketchart, K. (2003). “A Review of Case Histories on GRS Bridge-Supporting Structures with Flexible Facing.” Proc. 82nd Annual TRB Meeting.

[187] Zelada, G. A., and Stephenson, R. W. (2000). “An Evaluation of Auger Cast-in-Place Pile Design Methodologies for Compression Loading.” *Proc. of Int. Conf. on New Technological and Design Developments in Deep Foundations*, Geo-Denver 2000, ASCE, August 5-8, 2000, Denver.

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