APPLIED APPROACH SLAB SETTLEMENT RESEARCH, DESIGN/CONSTRUCTION

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1.0 INTRODUCTION

This annual report describes a summary of work accomplished during the first year of this two year project. The purpose of this research is *to investigate the causes of bridge approach settlement via literature review, direct investigation, and laboratory investigations and propose design and construction solutions.*

The study will critically examine current practices in the state of Oklahoma via direct forensic investigation of selected bridge approaches by field and laboratory investigations and an exhaustive literature review. The scope of this project includes two phases: a literature review and a critical investigation of practices via field and laboratory investigations.

2.0 SUMMARY OF TASKS

Table 1 presents a summary of the original proposed tasks.

Table 1 Summary of Tasks

Task Number	Description	Purpose
Phase 1 – Literature Review of Approach Slab Settlement Problems and Solutions		
1-1	Collect and Summarize Available Literature	Review state of design practices.
Phase 2 – Critical Investigation of ODOT Design and Construction Practices Related to Bridge Approach Settlement Problems		
2-1	Selection of Approach Slab Sites for Forensic Analysis	Identify test sites representative of typical problems.
2-2	Gathering of Background Information	Collect all available design and site information for selected sites.
2-3	Field Investigations of Sites	Conduct Visual Investigations and non-destructive testing
2-4	Laboratory Investigation of Soil Materials	Assess soil properties and behavior
2-5	Assessment of Forensic Data	Analyze the information gathered in tasks 1-1 through 2-4 using deductive reasoning and statistical analysis.
2-6	Recommendations to Minimize the Approach Slab Settlement Problem in Oklahoma	Develop recommendations based on knowledge gained.

3.0 PROGRESS

This section describes progress so far with regard to each of the seven major tasks associated with this project.

3.1 TASK 1-1: LITERATURE REVIEW

During the first year substantial literature has been collected and summarized that addresses causes of bridge approach settlement and possible solutions. The literature review as it stands so far is presented below. This is a work in progress and will require refinements and additions in the final report. This literature review demonstrates the wide variety of causes and solutions to addressing this complex and persistent problem. Many of the issues presented below for states other than Oklahoma, resonate with the issues faced in Oklahoma.

INTRODUCTION

A bump often develops in a bridge near the interface of the abutment and the approach. This is because the approach often settles more than the abutment (Abu-Hejleh et al., 2008). This problem causes uncomfortable and dangerous driving conditions and significant damage to bridges which require expensive repairs and traffic delays. It is estimated that 25% of U.S. bridges are affected by this issue (Mishra et al., 2010). Some of the most common causes include compression of the fill material, settlement of the foundation soil, poor construction practices, poor fill material, loss of fill due to erosion, poor joints, temperature cycles, and poor drainage. The most common remedies to date are the use of geotextiles, improved drainage and improved construction practices.

There have been numerous studies over the years focused on bridge approach slab settlement problems. One of the better known research documents is a NCHRP synthesis study published in 1997. In this synthesis of highway practice, Briaud et al. (1997) identified several causes for the settlement of bridge approaches that lead to the bump at the end of the bridge. According to Briaud et al. (1997), possible causes leading to bump problems include (1) seasonal temperature change (2) loss of fill material by erosion (3) poor construction practices (4) settlement of foundation soil and (5) high traffic loads as shown in Figure 1.

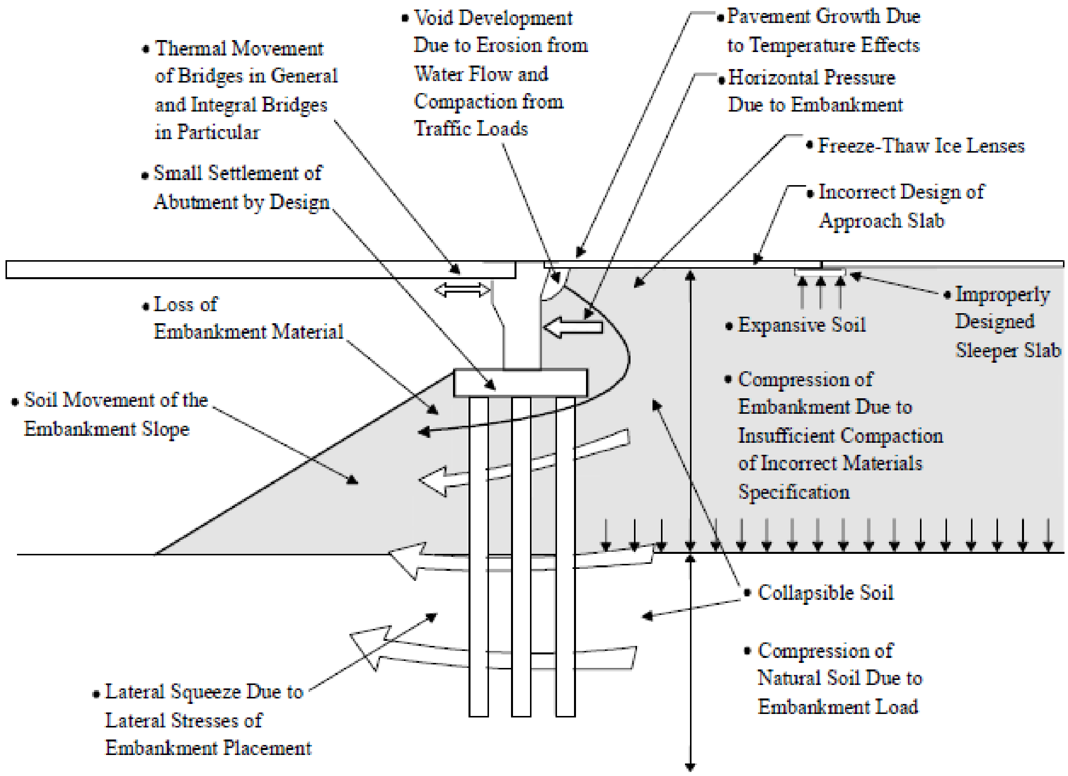

Figure 1 Problems Leading to the Existence of a Bump (Briaud et al., 1997)

In a field evaluation of 19 New Mexico bridges (Lenke 2006), it was concluded "that most, if not all, bump problems in the state are associated with geotechnical issues." These geotechnical issues were identified as primarily deep seated foundation problems and poor quality fill materials partly attributed to construction inadequacies. Erosion and drainage concerns were cited as contributing factors but of a secondary nature compared to geotechnical issues and geometry of approach slab.

In Iowa, White et al. (2005) investigated 74 bridges to identify the bump problems of bridges. According to his field investigation, the major contributing factors are characterized by the development of subsurface voids within one year of construction as a result of loss of backfill material due to erosion, water infiltration through unsealed expansion joints, high traffic loads, horizontal movements of integral abutments caused by seasonal temperature changes as well as having slopes higher than 1/200 which is a maximum acceptable gradient for bridge approach slabs. These bridge approach problems are shown below field observation pictures in Figure 2.

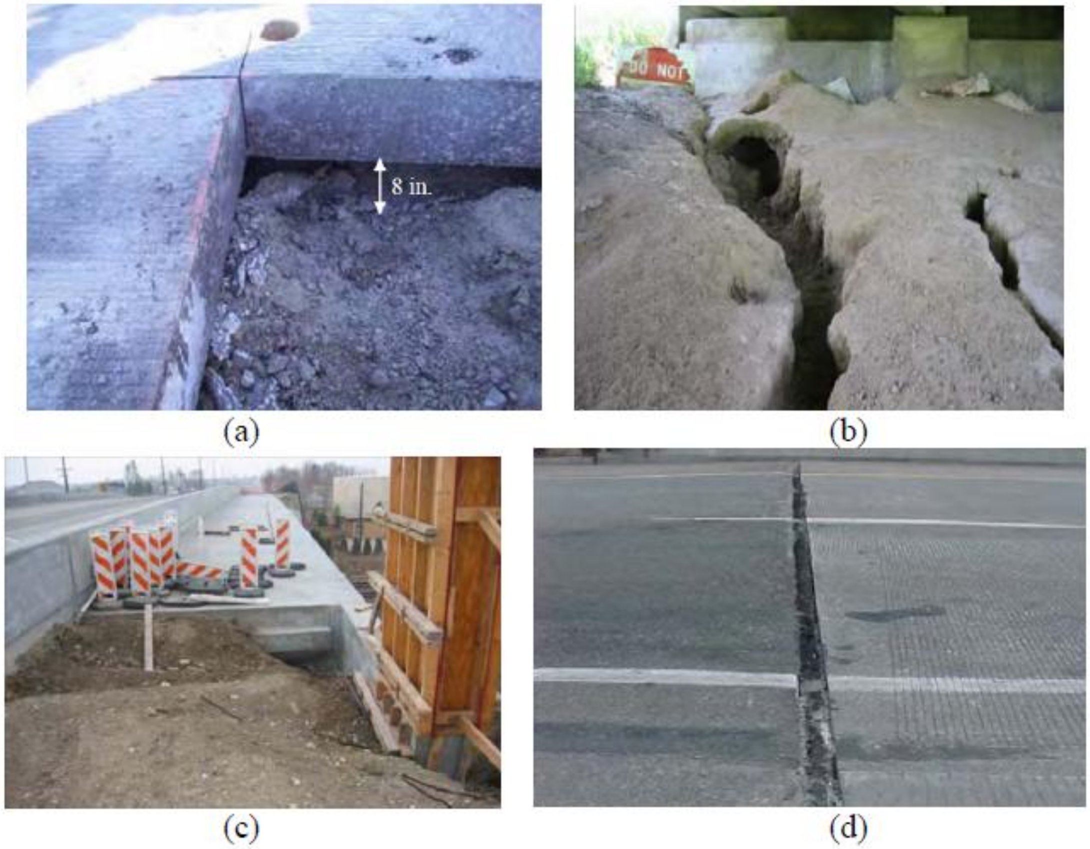

Figure 2 (a) Observed Void under the Approach Slab, (b) Soil Erosion of Granular Backfill, (c) Non-compacted Granular Backfill Behind the Bridge Abutment from Poor Construction Practice, (d) Unsealed Expansion Joint (White et al., 2005)

SETTLEMENT

Settlement under the approach slab can occur in the embankment or in the foundation soil. White et al., (2007) list major causes of approach settlement including: consolidation of the foundation soil, poor drainage and poor compaction of the embankment fill, and erosion around the abutment.

Settlement of the Foundation Soil

Various studies have investigated the relationship between embankment settlement and the foundation soil. Whals (1990) claims that the behavior of the foundation soil is one of the most important factors in the performance of a bridge approach. Immediate settlement of the foundation soil occurs at the application of the load and is not generally a problem; primary and secondary consolidations are time dependent and therefore significant factors in problematic bridge approach settlement. Problems arise when an embankment is constructed on soft soils because the foundation soil is subject to excessive settlement or stability problems caused by lack of bearing capacity (Pearlman and Porhaba, 2006). If stresses in the foundation soils approach the shear strength of those soils, they may be laterally displaced which can cause major distress to the embankment. Such issues are more typical in soft, cohesive soils than in cohesionless soils (Whals, 1990).

Karim et al. (2003) studied a bridge embankment that was constructed on soft, normally consolidated clay which was underlain by peat and sand layers. The study concluded that difficulties in settlement prediction arise due to uncertainty in precipitation and drainage performance.

In Nebraska, (Tadros and Benak 1989) prepared a field survey for 4 years at 16 different sites and reported foundation soil consolidation as fundamental factor leading to the formation of bridge approach settlement.

Wetting-Induced Collapse

Wetting-induced collapse settlement occurs when the post-construction moisture content is increased by events such as precipitation, capillary water from the foundation soils, or flooding. Four factors that have a significant influence in wetting-induced collapse problems are the soil type, total overburden stress, pre-wetting moisture content, and dry unit weight (Lim and Miller, 2004). In addition to these, Lawton et al. (1992), proposes that there needs to be a bonding or cementing agent that stabilizes the soil in the partially saturated condition which is reduced with the addition of water to the soil, causing the intergranular contacts to fail in shear and ultimately cause volumetric shrinkage of the soil mass. For any given compacted soil, pre-wetting moisture, dry-density, and stress-state are the most important factors in determining collapse potential.

Lawton et al. (1992) determined that nearly all compacted soil types can be susceptible to collapse under the right conditions. For sand-silt-clay mixtures, the collapse potential is greatest when the clay fraction ranges from 10% to 40% by weight. For sand-clay soils, the maximum collapse potential occurs with a clay fraction of 30% to 40% while for silt-clay soils the range is 10% to 20%. At lower clay contents, the silt collapses more than the sand while at higher clay contents, the silt collapses less or swells more. As the clay fraction increases and clay comes to dominate behavior, the silt and sand behavior seem to merge in relation to the clay behavior. The researchers hypothesize that at low clay content, the clay acts as a binding agent between the silt or sand particles and that when wetted, the clay binders become soft and lubricate the intergranular contact of particles and facilitate collapse. As silt particle sizes become smaller, they become more flat, like clay, and the fabric becomes less stable and the collapse potential increases.

Moisture changes will occur in all portions of a fill with enough time, but it is near the surface that the most significant changes occur. Studies show that, in general, collapse potential increases as water content and density decrease, and vertical stress increases (Lawton et al., 1992). The drier a soil is during compaction, the greater is the potential for collapse. This is because a soil compacted near to or wet of the line of optimums has already undergone much of the potential wetting-induced collapse. Increasing the relative compaction substantially reduces the wetting-induced collapse, increases the critical overburden pressure at which collapse is maximized, and increases wetting-induced swelling. Therefore, a balance must be sought between collapse and swell when specifying compaction. The concept of critical moisture condition is valid if expressed as the degree of saturation instead of water content (Lawton et al., 1989). Lawton et al. (1992) argue against the idea that a soil compacted wet of optimum cannot collapse because this ignores the prewetting density and the possibility for the soil to dry post-compaction. Lawton (1986) tested two clay samples. Both had a water content of 16%, one was compacted to a relative compaction of 89%, the other 100%. The sample compacted to R_s=89% collapsed 3.4% while the sample compacted to R_s=100% exhibited negligible collapse. Lawton claims that the higher degree of saturation in the second sample is most responsible for reduced collapse potential.

Collapse potential reaches a maximum at some critical value of vertical stress; however, in many cases this value is so large as to be meaningless. This reduction of collapse potential at vertical stresses above the critical stress is caused by the densification and increased degree of saturation resulting from the applied stress. (Lawton et al., 1992).

Lim and Miller (2004) performed oedometer tests and found that collapse occurred over a wide range of dry unit weight. Cohesionless sandy soil exhibited negligible collapse potential while clayey soils showed more significant collapse potential for soils compacted at 95% relative compaction and dry of the OMC (based on standard Proctor tests). An issue discussed in Lim and Miller's research is that even collapse potential defined as "slight" and having a 2% vertical strain can still lead to significant settlement as embankment height increases. Their laboratory results were compared with two bridges and one centrifuge test. They concluded that typical compaction specifications specifying a minimum relative compaction of 95% and moisture content of OMC ±2% are not always satisfactory because they do not account for the significant collapse potential that can occur in compacted fill embankments. They suggested that specifications could have more stringent compaction requirements and increased quality control to reduce collapse settlements.

Nondurable Material Causing Settlement

Durable rocks do not slake in the presence of water; however, nondurable rocks do. When used in a bridge embankment, nondurable rocks are subjected to compressive forces and slaking when water is present. When these nondurable rocks slake, they become soil that fills in the voids between rocks and cause settlement. It is recommended that laboratory testing be done to predict the settlement effects from slaking nondurable rocks. For embankments made of sedimentary rocks, the soaked compression test has been used. (Vallejo and Pappas, 2010).

Poor Drainage Causing Settlement

Drainage plays an important role in the settlement of cohesive soils. Approach drainage falls into two major categories: directing surface water away from the embankment and removing infiltrated water behind the abutment (White et al., 2007). Improper surface drainage allows water into the backfill, and inadequate or non-functioning subsurface drainage is destructive to the approach. Water that gets between the abutment and the approach can erode the backfill without proper drainage. Three typical subsurface drainage systems are: granular and porous backfill around a perforated drainage pipe, the addition of a geotextile to the previous system around the porous fill to reduce fines infiltration and erosion, and geocomposite drains to provide pathways to the drain tile (Mekkawy et al., 2005; White et al., 2007).

In another study, Ha et al. (2002) after a detailed study of two bridges in Houston reported that the likelihood of being exposed to the water is greater for the soil near abutment than the soil far from abutment. According to his investigation in this situation, having lower strength and higher compressibility in soil near abutment leads to bridge approach slab settlement or bump at the end of the bridge.

EROSION

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

Piping

Piping is the interaction of fluids and solids where flowing water creates a drag force that carries soil particles (Liang et al., 2011). The cavities formed in this process can become large and collapse (Sinco et al., 2010). Adams and Xiao (2011) proposed mixing organic soil with sand to increase the sand's resistance to piping. In their investigation, they studied the use of organic soil to reduce susceptibility to piping and whether the organic mixes would be suitable for use as a backfill material. The investigation used sandy-soil from a construction stockpile and commercially available "compost" consisting of an equal proportion of green-waste and bio-solids. The mixed the organic material into the sand in the proportions of 5%, 10%, 15%, and 20% by mass. Their investigation found a positive correlation between organic content in the soil and the reduction of erosion. Regarding compressibility, the 20% mixture saw an increase of 140% in settlement from the pure sand, and was therefore found to be unsuitable for use as embankment backfill material.

Suffusion of Granular Soils

Suffusion is a process where water seeping through a granular soil dislodges fine particles without destroying the coarse grain soil matrix. Such soils are called "internally unstable." Suffusion occurs when soil has a specific structure and primary fabric and particles which are smaller than the constriction of the primary fabric. These smaller particles fill the voids of the primary fabric. When a flow is present, it can transport the loose particles away through the voids. The fines in these matrices are in the voids and may not support effective stress. (Indraratna et al., 2011; Shwiyhat and Xiao 2010). Internal erosion can be very harmful to earthen structures. While suffusion is less catastrophic than piping, it can lead to increased permeability, seepage, and consolidation of a soil layer (Wan and Fell, 2008; Shwiyhat and Xiao 2010). Shwiyhat and Xiao (2010) conducted an investigation into the effects of suffusion on settlement potential of a soil layer due to the loss of fines from the coarse grain matrix. Two important conclusions from their tests are that the suffusion of fine particles may clog the downstream soil matrix and reduce permeability and that specimen volume decreases with internal erosion. The volumetric changes were minor and occurred early in the testing.

Laboratory tests have shown that internally unstable soils will begin to erode at a hydraulic gradient less than the critical gradient. Wan and Fell (2008) found that many samples began to erode at gradients from 0.8 to less than 0.3 with the trend being that soils with higher porosity eroded at lower gradients. Ahlinhan et al., (2010) investigated the effects of critical gradients in both horizontal and vertical flow on soil gradation. For suffusion to occur, there are both geometric and hydraulic criterion. The geometric possibility is based on particle size distribution. Ahlinhan et al. sought to define the hydraulic criterion by testing five different non-cohesive soils in upward and horizontal flow conditions. In the upward flow, the critical flow for stable soils was close to the theoretical value as determined by the Terzaghi and Peck (1961) equation for critical hydraulic gradient and ranged from around 0.7 to 0.9; for the unstable soils, critical gradients of 0.18 and 0.23 were measured. The horizontal flow experiments found a dependence of the critical gradient on the sample's relative density.

EMBANKMENT HEIGHT

Load transfer is directly related to the embankment height. In a numerical model analysis by Hello and Villard (2009), it was found that the efficacy of the transfer of load increased with the height of the embankment and the results stabilize for the higher embankments. The load resistance of the embankment increases as the height decreases (El-Naggar and Kennedy 1996). In an investigation on the use of piles, settlements increased with embankment height (Chen et al., 2010).

In Oklahoma, bridge approach settlement is a serious problem. Laguros et al. (1990) performed a comprehensive study of bridge approach settlements in Oklahoma and found that a number of embankment, foundation and bridge features were statistically correlated with the magnitude of observed settlement. The study, involving a survey of 758 bridge approaches, of which about 83% experienced settlement, suggested that the two most statistically significant factors were the age of the approach and the height of the embankment.

In 2003, Seo performed a wide questionnaire survey on bridge approach settlement problems among 25 districts of Texas DOTs. The survey results show the possibility of having serious bump problems when embankment is high and the fill is clayey soil.

PRACTICES TO ALLEVIATE SETTLEMENT OF THE FOUNDATION SOIL

When addressing settlement of the foundation soil, there are two main options: transferring the embankment loads through the weak layers to more competent strata or improving the foundation soil (Wahls, 1990). Farnsworth et al. (2008) investigated three methods for timely construction on soft foundation soils and found that an expanded polystyrene (geofoam) embankment with tilt-up panel fascia walls performed best. The primary difficulty encountered in this investigation was limiting primary consolidation on soft foundation soils.

Piles

Using piles to support embankments over soft ground has demonstrated advantages such as rapid construction, small lateral deformation and settlement control. In the investigation of three pile-supported embankments, the arching height to reduce load on soft soils and transfer it to the piles and competent bearing layers was between 1.0 and 1.5 time the net pile spacing. (Chen et al., 2010). Beeker et al. (2005) conducted a study of a large number of pile-supported approach slabs in southeastern Louisiana. Using the Louisiana Department of Transportation and Development rating system for approach slabs, they found that pile-supported slabs typically had acceptable ratings. Seven representative bridges were selected for an in-depth investigation. It was concluded that there was a wide range of performance of pile-supported slabs in Louisiana and the inconsistencies were largely due to differences in negative skin friction from site to site.

Deep Soil Mixing

Archeewa et al., (2011) at the University of Texas investigated the use of deep soil mixing (DSM) to mitigate differential bridge approach settlement due to soft foundation soils. This study involved construction of a bridge in North Arlington, Texas. One side of the bridge utilized DSM columns while the other was constructed using conventional techniques. Settlement was measured both in the foundation soil and the embankment, giving a total picture. The DSM technique consists of a column which is cut into the soft soil by rotating blades and cementitious compounds that are forced into the column and mechanically mixed with the soil. The cementitious compounds vary, but the underlying principle is to improve weak subgrade soils (Archeewa et al., 2011; Lin and Wong, 1999).

An important consideration when using the DSM technique is the area ratio (a_r). This is defined as the ratio between the treated area and the total unit area of soil, which is the summation of both treated and untreated plan area (Archeewa et al., 2011). In the study, settlement reductions began to plateau around an a_r of 0.6. After that point, foundation settlement became negligible and total settlement was dependent on the embankment settlement. Lin and Wong (1999) concluded that use of DSM techniques in two bridge approaches on soft clays significantly reduced foundation soil settlement.

Controlled Modulus Columns

The controlled modulus column (CMC) method to support structures such as embankments consists of using a specially designed augur to displace soil laterally without vibration, then developing a column by pressure grouting during augur extraction. According to Pearlman and Porbaha (2006), this technique improves soft foundation soil by increasing the densification and reinforcing it. No soil mixing takes place during the pressure grouting. Pearlman and Porbaha's investigation was conducted by implementing CMC in very soft alluvium foundation soil beneath a 7.5m high embankment. The study concluded that the CMC foundation was useful to provide a timely foundation solution.

Miao et al., (2009) conducted an experiment on one cement grout mortar CMC in soft clay as well as unimproved soil for comparison purposes. The study concluded that CMC can reduce settlement significantly. In order to investigate the effect of the treatment, two types of plate loading tests were conducted on composite foundations as well as a test on untreated soft clay using a rigid plate. The composite specimens had a single column in the center; one had a flexible loading plate and the other had a rigid plate. Both treated samples settled significantly less than the untreated soft clay. The stress concentration ratio of column to soil in the rigid was about 2.0 and in the flexible about 5.0. From this, the researchers concluded that the cushion stiffness on the CMC can be changed to control stress concentration ratio. As well, it was concluded that the CMC technique offers greater control and consistency than soil-cement columns.

PRACTICES TO IMPROVE THE EMBANKMENT

Geotextile and Geogrid Reinforcement

Bergado et al. (1994) investigated the use of geotextile reinforcement on bridge foundations by constructing two full scale embankments. A planned 6 m-high embankment was reinforced with four layers of low strength, non-woven reinforcement at the base; this embankment failed after reaching 4.2 m. Another 6 m-high embankment with one layer of high-strength woven-nonwoven geotextile was finished and tested to failure. A 4 m control embankment was constructed without reinforcement. The site consisted of 2 m of weathered clay, 6 m of soft clay, 2.5 m of stiff clay with sand lenses and silt seams, and 3.5 m of stiff clay. Settlement was measured at depths of 0, 2, 4, and 6 m from the original ground surface. The results of this study were that the geotextile reinforcement increased the factor of safety by nearly 60% in the ultimate height of the unreinforced embankment. They concluded that the use of high strength geo-textiles at the base can considerably increase the ultimate height of an embankment on soft clay. According to Edgar et al. (1987) the use of geotextiles by the Wyoming Highway Department (WHD) has reduced the lateral and vertical deformation of the approach embankment and significantly reduced the differential settlement between the approach and abutment. When geotextiles are used to transfer loads to piles using the membrane effect, the stiffness of the geosynthetic directly affects the behavior (Hello and Villard, 2009).

Mechanically Stabilized Earth And Lightweight ECS as Fill

The Colorado Department of Transportation (CDOT) has employed three alternatives to alleviate the bump at the end of the bridge, including flowable fill, mechanically stabilized earth (MSE) with well graded granular Class 1 backfill, and MSE with free-draining Class B filter material. However, the bump problem has persisted (Abu-Hejleh et al., 2008). CDOT then commissioned a study to improve the effectiveness of the procedure and to research means to make the procedure more economical. In an investigation using three MSE walls along two-lane highway embankments, settlement and piping caused voids and roadway distress. The sites had 45% fines in the backfill material and surface water was allowed to flow through the clean gravel roadway base course to the MSE wall. Preventing water infiltration therefore is important in an MSE wall and preventing piping of the backfill material (Dodson, 2010).

Research was performed on the usage of light weight aggregate (LWA) material produced from expanded clays and shale (ECS) for its potential as fill material to reduce approach settlement. The production of ECS aggregate is strictly controlled to ensure uniformity. In a recent study (Saride et al., 2010), it was demonstrated that the material could reduce dead weight and resulted in high internal stability. The high internal friction angle reduced vertical and lateral forces. Compressibility and swell tests were performed to characterize volume change properties according to ASTM D2345. In these tests, the ECS was compared to typical fill materials (fine sand-sand and gravel) and was found to have a lower bulk density by half, higher friction angle, and significantly lower compression index (C_c).

In addition to material tests, an embankment was constructed over 6-m-thick soft moist clay layer underlain by a 3-m-thick layer of dense sand underlain by a hard sand stone. The Texas Department of Transportation used LWA material on the approach to reduce the load on the foundation soil on one end of the bridge, while the other end was constructed with normal embankment material. Proper compaction of the ECS material was a challenge due to the higher volume of voids. The control end of the bridge was built in several lifts and compacted to a relative density of 98%. The ECS embankment performed satisfactorily and can be used to alleviate the bump at the end of the bridge. One issue that came up in the experiment was localized bulging on the slope induced by rainfall (Saride et al., 2010).

FIELD COMPACTION

Quality control (QC) and quality assurance (QA) are typically based on density measurements. They often ignore mechanical properties. The depth of influence of density measurement (300 mm) may not be adequate to detect deeper weak spots. (Gallivan et al., 2011).

Intelligent Compaction

Intelligent compaction technology provides greater process control during the compaction process. The Federal Highway Administration (FHWA) defines intelligent compaction as a process where densification is achieved by the use of a double-drum vibratory roller equipped with a measurement /control system that automatically controls the compaction parameters in response to measured material stiffness. The roller must also be equipped with documentation devices that provide a continuous record of roller location and the corresponding output including information such as the number of passes and material stiffness measurements (Gallivan et al., 2011). This procedure is expected to be generally applicable to cohesionless sandy soils susceptible to compaction by vibration.

Dynamic Compaction

Dynamic compaction is a technique wherein a large mass is dropped from a pre-determined height to densify the soil. Dynamic compaction has been used on many soil types and primarily in sandy materials and granular fills (Mostafa and Liang, 2011; Feng et al., 2011). Feng et al. (2011) conducted an investigation into the field effectiveness of dynamic compaction using various methods. They used standard-penetration and surface wave tests to evaluate the depth of improvement. Conclusions of this study were that dynamic compaction was effective in improving loose medium-grained granular soils, correct energy input was imperative, and time is required between each pass for excess pore water pressure to dissipate.

SUMMARY OF LITERATURE REVIEW TO DATE

The forgoing literature suggests that there are multiple causes to bridge approach settlement related to the bridge design and design of the drainage system, quality control measures, embankment geometry, bridge type, and properties of the embankment and foundation soils. Numerous potential solutions exist depending on the expected problems. The current study aims to highlight typical problems associated with Oklahoma bridges and provide recommendations for alleviating these problems before and after construction.

3.2 TASK 2-1: SELECTION OF APPROACH SITES

The current research will focus on several sites where bridge approach settlement has occurred with the aim of understanding and identifying the primary factors causing the settlement. Site selection has been carried out via a multi-stage approach. The first stage is identifying potential sites. This has been accomplished in two ways in the first year. First, the PIs met with ODOT personnel and 12 potential sites were identified. After this, a survey was sent to division engineers throughout the state. As of this report, a total of 49 bridges have been identified as potential sites for further investigation.

The second stage of site selection involves a preliminary visit by the research team. Preliminary visits are comprised of a visual inspection of each approach, making measurements to estimate settlement and other deformations where it is safe to do so, and taking photographs. The visual inspection makes note of the condition of the approach slab, abutment, embankment, wing walls, drainage, and other peripheral structures for evidence of the causes of settlement and related issues. Measurements are made of the vertical and horizontal differences between the approach slab and the bridge deck and wing walls, the depth of voids, and other significant clues. To date, the team has visited 20 bridges.

After each preliminary visit, the field team presents pictures and commentary to the entire research team at the weekly meetings. The team discusses each site as to possible issues, general commentary, and whether a site has potential for further research.

The third stage of site selection is comprised of critically examining the results of the preliminary visits and collecting and analyzing bridge design and background information. After analyzing the available information and conditions of the bridge during the inspection, the team selects sites based on severity of the issue and various design parameters in order to investigate sites with a variety of issues and design styles. To date, the research team has designated 9 bridges for in-depth analysis with the possibility of future additions. The top nine sites identified for further study are shown in Table 2. Four of these sites have been analyzed in some detail as described below.

Table 2 Summary of Top 9 Selected Sites

Location & Bridge	NBI	County	Embankment (ft.)	Built in	(Non) Integral	Foundation Soil
US 177 over The Arkansas River; Bridge A	24475	Noble	30' (South) 10'(North)	1996	Non-Integral	Predominantly Clays with PIS from 20s to 30s
Tecumseh Road over I-35	24822	Cleveland	25'	1997	Non-Integral	Fat Clay mostly; approximately 10' thick
SH-6 over Sadler Creek; North Bound	26401	Beckham	No Embankment, few feet of fill	1999	Integral	Approximately 70': Sandy lean clay near surface and lean clay deeper
SH-1 over SH-99	26915	Pontotoc	10'; 20'	2001	Integral	Silty Clay, Clayey Shale, 10'
SH-59A over Big Creek	24277	Pontotoc	5-10' fill	1996	Integral	Approximately 50' of silty clay, sandy clay, and clay.
Shields Boulevard over I-35	27771 & 27772	Cleveland	15'-25'	2004		Clayey Shale
Hereford Lane over SH-69	27769	Pittsburgh	25'	2008 (original); 2011 (reconstructed approaches)	Integral	Lean to fat clay
SH-11 over I-35	27771	Kay	15	2007	Integral	Clay
SH-6 over West Elk Creek	25128	Beckham	2'	1999	Integral	Silty sand, crushed sandstone

US 177 OVER THE SALT FORK OF THE ARKANSAS RIVER, BRIDGE A NBI 24475

Bridge A of the US 177 over Salt Fork site one of a 3 bridge site that has been part of a previous study by the Oklahoma State University which occurred in the mid-1990s. The bridge was completed in 1996. The soils at bridge A are predominantly clays and silts with PIs ranging from 20s to 30s. The embankments for bridge A were constructed using typical procedures. An unspecified backfill was compacted to the specified densities (Snethen et al., 1997).

Embankment Construction

The north embankment of the A bridge was constructed using silty sand (SM, A-2-4). The height of the north embankment is 9.8ft (Koeninger, 1997), while the south embankment is approximately 30-35' thick according to the elevations found on ODOT plans. The compaction method was a Case 1150C tracked front end loader passing over 1 ft. thick lifts with a full bucket. The loader passed over each lift twice, once parallel to the abutment wall and once perpendicular. Within two feet of the abutment and wing walls, compaction was achieved using a walk behind pad vibrator. Drainage for each abutment is provided by using a perforated PVC pipe placed along the base of the abutment which was covered with a granular material (Koeninger, 1997). Embankment A-1 can be seen in Figure 3. The abutment is a non-integral abutment.

Figure 3 US 177 Embankment A-1

Instrumentation of the Oklahoma State University Research

Inclinometers were used in each abutment to measure lateral movement and settlement of the backfill and the foundation separately. Amplified liquid settlement gauges were also used to measure settlement. Open tube piezometers were used to measure the groundwater levels. Total pressure cells were installed on the abutment wall to measure the lateral earth pressure; each wall has pressure cell at the top, middle, and bottom. Surface settlement was also tracked using survey points. The OSU study only partially instrumented embankment A1, as it is taller than the rest. It lacks the west-of-center inclinometer and west-of-center amplified liquid settlement gauge (Koeninger, 1997).

Results of the Oklahoma State University Research

Embankment A2 (North)

The top total pressure cell peaks at 1.3 psi, with the pressure peaking during the summer and the minimum during winter. Koeninger, (1997) believes that this is due to the bridge deck expanding during the hot summer months. The middle cell has a peak pressure of 1.6 psi, with a similar trend as the top, but lower differential pressure. The bottom cell pressure rose immediately after construction to 2.2 psi and had then decreased steadily over the course of the investigation to a value of 0.3 psi. The investigators expected the lateral earth pressure distribution to increase pressure with depth and suggest that arching of the granular material could be responsible for the low pressure value.

The magnitudes of lateral earth movement of the backfill and abutment wall were small at the time of the investigation, ranging from 0.12 inches at the top of the abutment wall casing to 0.4 inches in the top of the offset casing. Parallel to the face of the abutment wall, the centerline and offset cases have moved east. The offset casing had moved 0.12 inches. The abutment casing had moved 0.05 inches west toward the wing wall (Snethen et al., 1997).

Measurements of the amplified liquid settlement gauges show the majority of settlement occurring within the first ten months after construction and then leveling off. At the time of the study, the centerline had settled 0.332 ft. (3.94 inches) and the offset 0.247 ft. (2.96 inches). Their readings indicated settlement below the backfill, in the embankment and foundation soils. Surface settlement measurements show 0.03 ft. of settlement in the centerline wheel path 5 ft. behind the abutment and 0.061 ft. 20 ft. behind the abutment. In general, the settlement is increasing with distance from the abutment wall. The telescoping inclinometers settlement at the time of the research is presented in Table (Snethen et al., 1997). At the time of this study, a “bump” was developing at this embankment.

Table 3 Embankment A2 Inclinometer Results

	Centerline	Offset
Backfill	0.005 ft. (0.06 in)	0.075 ft. (0.9 in)
Embankment	0.001 ft. (0.012in)	-0.005ft (0.06in)
Foundation	0.074 ft. (0.89 in)	0.085 ft. (1.02 in)

Activities of the University of Oklahoma Team

Karim Saadeddine and Colin Osborne made a preliminary site inspection of the three US 177 bridges on June 2nd, 2011. The team made a visual inspection of the site and measured surface differences and other points of interest via hand tools.

Embankment A-1

A vertical difference between the bridge slab and current asphalt level was 1.5" and the horizontal difference between the approach slab and wing walls was 0.5". Maintenance has been performed on this bridge. New asphalt has been poured over old asphalt, and the new asphalt is also cracked and distressed. Current conditions can be seen in Figure 4.

Figure 4 Karim Saadeddine Measures 2" Vertical Difference at Interface. Asphalt overlays visible.

Embankment A-2

The piles under the A-2 abutment are exposed up to 6-7" on one side. It appears that water is coming down the abutment in this area, but the drain is dry. This could suggest drainage issues. Maintenance of asphalt patches has been applied to this side as well. Figure 5 presents a view of the approach. Figure 6 shows soil voids under the abutment.

Figure 5 US-177 Approach A-2

Figure 6 Erosion Under Abutment A-2, US-177 over Salt Fork

TECUMSEH ROAD OVER I-35

NBI 24822

This site is in the Norman, Cleveland County, Oklahoma. The bridge at this site carries Tecumseh Road over I-35. The bridge is 210' long, was finished in 1997, carries both directions of traffic and is non-integral. The average daily traffic for this bridge is 74,900 with a truck percentage of 12%. Both approach embankments are approximately 25' above the existing ground line. Each approach is 40' long.

Embankment & Approach Construction

Approach and Embankment

Each approach is 40' and comprised of two reinforced concrete 20' slabs. The backfill material is a granular backfill and the embankment is constructed from unclassified borrow soil.

Drainage

Drainage for the abutments is provided by a 6" diameter perforated pipe behind the bottom of the bridge seat. The pipe surrounded by a 1' square coarse pipe underdrain cover material which is then surrounded by a 2' (including the coarse material and pipe) square filter sand pipe underdrain cover material.

Geology and Site Information

According to the foundation report, the interpreted rock line is approximately 35' below the road surface. The rock is described as reddish-brown shale. The soil layer is approximately 10' thick and consists mostly of fat clay. Boring 2 is close to Approach No. 1 (west end of bridge), Boring 3 is in the median of I-35, and Boring 1 is in the middle of the northbound I-35 highway. The existing groundline is fairly level across the site and the three borings are relatively uniform. The water table was located about 5' below the ground on the date of borings. The bedrock consists mostly of shale.

Approach Condition

In the October 7th, 2009 Bridge Inspection Report, the approach slabs have up to 3" of settlement, 3" undermining at the edges, and the seals had been torn loose. The OU research team visited this site on Friday, September 16th, 2011. The team noticed substantial voids that had been filled under the approach slabs, especially on the west end. The team also noticed erosion under the slope wall east of I-35.

Figure 7 Horizontal Displacement; Looking West Across Tecumseh Road Bridge

Figure 8 Void Beneath Approach Slab

Figure 9 Maintenance and Distress Under Approach Slab

Figure 10 Drains and Voids At Slope Wall

Likely Causes

The height of the embankment makes collapse of the fill a potential issue; evidence of erosion under the slope wall suggests that erosion could also be a factor in the approach settlement. Settlement of the foundation is also possible with 10' of fat clay.

Recommendations for Further Investigation

We recommend sampling both the embankment material and foundation material under the approaches to classify and characterize the subsurface and also obtaining Shelby tube samples away from the embankment to get information on the foundation soil behavior.

STATE HIGHWAY 1 OVER STATE HIGHWAY 99

NBI 26915

The SH-1 over SH-99 Bridge is an integral bridge constructed in 2001. It is built on silty clay and clayey shale soil. The bridge is a 2-span, 272' bridge crossing over a state highway near northwest Ada, Oklahoma. The average daily traffic is 11,800 with a truck percentage of 7%. Following is a summary of preliminary information on this site and recommendations for further study.

Embankment & Approach Construction

Approach No. 1

Approach 1 is the western end of the bridge. This approach slab is reinforced concrete. The road elevation here is approximately 10’ above the existing groundline. The road-end of the approach has no skew, the bridge-end of the approach has a 10° skew. The length of the approach is 30’ on the short side and 40’ – 4 7/8” on the long side. The backfill material is granular with a CLSM layer between the backfill and concrete slab. There is a one inch sealed joint between the approach slab and the approach slab and the wing wall. This approach slab consists of 6 slabs. Longitudinally, there is a 2” rapid cure silicone joint 9’ 10” from the road. Transversely, there is a ½" sawed and sealed joint 24’ 6” from the outside edges of the slab, creating a 10’ region in the center of the approach. The joint between the approach and the bridge deck is ½” sawed and sealed joint with ¼” chamfers.

Approach No.2

Approach No. 2 is the eastern end of the bridge and is a reinforced concrete slab. This slab lies approximately 20’ above the existing ground line. There is no skew between the road and the approach slab, and a 10° between the approach and the bridge deck. The backfill material is granular with a CLSM layer between the backfill and concrete slab. This approach consists of 2 slabs. A 9’ 10” rectangular slab at the road end of the approach is joined to the rest of the slab by a 2” rapid cure silicone joint. The entire approach is 30’ on the short side and 40’ 4 7/8” on the long side.

Drainage

Drainage for the abutments is provided by a 6” diameter perforated pipe behind the bottom of the bridge seat. The pipe surrounded by a 1’ square coarse pipe underdrain cover material which is then surrounded by a 2’ (including the coarse material and pipe) square filter sand pipe underdrain cover material.

Geology and Site Information

Soil

Preliminary soil profiles are based on the ODOT provided foundation report found on Sheet No. A75 of the project plans. The three borings for this site were taken under the bridge portion. On the west end of the bridge, there is approximately 10’ of silty clay in the boring closest to Approach No. 1 (B-1). Beneath the soil there are interchanging layers of clayey shale and sandstone. Boring 3 shows a much shallower (2’) layer of clay. Under the soil in this boring is approximately 1’ of weathered clayey shale and then 12’ of sandstone and then approximately 20 feet of clayey shale, then a foot of sandstone before the boring ends.

Approach Condition

According to the June 24th, 2010 bridge inspection report there is minor leaching around some of the integral beam connections. The southeast approach was mudjacked in May 2010. Cracks are present in the deck and some have been sealed. The slab itself is in fairly good condition; however, the maintenance evidence suggests underlying problems. The mudjacking filled a void under the approach slab.

Figure 11 Approach Slab

Figure 12 Maintenance Under Approach Slab

Likely Causes

Due to the size of the embankments and the underlying soil, the potential exists both for embankment collapse and foundation settlement. No evidence of erosion was visible, but the possibility cannot be excluded.

Recommendations for Further Investigation

We recommend a level 2 investigation of this site. We recommend sampling both the embankment fill material and the foundation soil. General classification tests should be run on both the embankment and foundation soil. Compaction, oedometer and collapse potential tests should be run on the embankment soil and settlement investigations of the foundation soil should be run. Construction records regarding the actual compaction and quality control should be obtained and compared to our laboratory results. The existing borings were conducted under the bridge deck and therefore may not represent the material under the approaches very well; therefore at least two borings should be made at each approach to get a representative description of the underlying soil.

STATE HIGHWAY 59A OVER BIG CREEK

NBI 24277

The site is located in Pontotoc County. The bridge carries SH 59A over Big Creek. The bridge was constructed in 1996 and is 184' long, carries both lanes of traffic, has integral abutments, and has an ADT of 1100 (2008) with a truck percentage of 15% (2002). Each approach is 30' long and is comprised of two slabs. The approaches are constructed on 10' of fill and approximately 50' of foundation material, predominantly sandy clay. Bedrock at the site is mostly shale with weathered shale between the soil and rock.

Site Information

Geology

According to the bridge foundation report (Sheet No. 16), the bedrock geology is Pennsylvanian shale of the Pontotoc Group. In the boring logs, a layer of weathered shale, approximately 5-10' thick, lies between the soil and the competent shale. There are also lenses of sandstone in the shale layers.

Foundation Soil

Approach No. 1

The soil beneath the bridge approach fill material is about 15' of sandy and silty clays, 20' of clay, and 10' of silty clay.

Abutment No. 2

The soil beneath the bridge approach fill consists of 10' of sandy and silty clays, 6' of fine sand, 20' of clay, and 4' of sandy clay.

Fill Material

The fill material listed in the foundation report is described as clay and weathered sandy shale.

Embankment Construction

Approach

Each approach is 30' long and comprised of two slabs. A 1 ½" rapid cure silicone joint joins the two slabs in the center of the approach. The approach slabs are constructed of reinforced concrete and are constructed on CLSM over granular backfill.

Abutment

The bridge uses integral abutments. The wing-walls extend 11' from the abutment.

Drainage

Drainage for the abutments is provided by a 6" diameter perforated pipe behind the bottom of the bridge seat. The pipe surrounded by a 1' square coarse pipe underdrain cover material which is then surrounded by a 2' (including the coarse material and pipe) square filter sand pipe underdrain cover material.

Activities of the University of Oklahoma Team

Karim Saadeddine, Yewei Zheng, and Colin Osborne made a preliminary site inspection of 59A over Big Creek on August 26th, 2011. The team made a visual inspection of the site and measured surface differences and other points of interest via hand tools and took photographs (presented in the appendix).

Condition of the Site

Both ends of the bridge have asphalt poured on top of the approach slabs to maintain a smooth ride. There was a void between the slab and the earth at Approach No. 1.

The most recent inspection date is May 17th, 2010. According to this report, the bridge deck has had some grinding for rideability, cracks on the faces of the abutments due to approach slab settlement have been repaired, the approach slabs have settled 3-4".

The conditions at the driving surface are severe and have required maintenance during the life of this bridge. The underlying issues are also severe, especially the void area beneath the slab.

Figure 13 Overview of Site

Figure 14 V-Shape Deformation of Approach No. 1

Figure 15 Void Under Approach Slab

Figure 16 Asphalt Over Concrete, Approximately 2" Thick

Recommendations for Further Study

We recommend a full investigation of this site. The 10' of fill material provides ample opportunity for significant collapse; even at 1-2% collapse potential results in 1-2.5" of settlement. The foundation material is also prone to settlement.

Therefore, we recommend sampling both the fill and foundation material to run a full set of laboratory investigations for classification, fill collapse, and foundation soil stress history and settlement potential.

Construction and quality control records should be obtained, especially regarding the fill material to compare collapse potential results to in situ compaction data.

SUMMARY OF SITES SELECTED FOR IN DEPTH STUDY

The preceding discussion highlighted four sites of potential interest for more in depth study. These sites represent different bridge styles (e.g. integral versus non-integral), varying approach embankment heights, and potentially different causes of settlement. In addition to the drilling and sampling recommended above, additional testing at these bridges may include cone penetration testing, coring in the approach slab, ground penetrating radar, and possibly other tests as appropriate. The team will coordinate with the ODOT Materials division to discuss the sites selected for in depth study and plan additional field work in November 2011.

3.3 TASK 2-2: GATHERING OF BACKGROUND DATA

The research team has gathered plans and design sheets for 15 bridges, including all 9 currently selected for an in-depth investigation. Soil surveys have been collected for all 9 in-depth sites. Basic traffic data (2008 ADT) has been collected for all 49 bridges from the ODOT website. In addition, the team is collecting historical weather data from weather stations near each site in addition to available regional and local geologic information.

3.4 TASK 2-3: FIELD INVESTIGATION OF APPROACH SLAB SETTLEMENT SITES

This task is really comprised of two sub-tasks: preliminary visits, which have been described above, and in-depth visits. Subsurface investigations will be made at the sites selected for an in-depth investigation where there are not already existing ODOT subsurface explorations. To date, the research team has conducted hand auger investigations on 3 approaches (2 bridges). Where appropriate, the research team will work with ODOT to perform more advanced in-situ testing and sampling. Boring logs from the tested sites can be found in Appendix A: Field Logs on page 48.

In addition to forensic investigations of bridge sites, the team also visits bridges under construction to study construction techniques.

Hereford Lane over SH-69

The team has made two visits to the Hereford Lane over SH-69 construction near McAlester, OK.

Visit One

The team first visited this site in August 2011. On this day, the demolition crew was tearing out the existing approaches. The following photographs are from this visit.

Figure 17 Hereford Lane East Approach: Pre-Demolition

Figure 18 Hereford Lane West Approach: Demolition

Visit Two

The team revisited the site in early September to watch the new rammed aggregate pier installed. This is a novel approach being used by ODOT. The following photographs were taken during this construction.

Figure 19 Aggregate

Figure 20 Drilling

Figure 21 Placing and Vibrating Aggregate

Figure 22 Testing Pier

3.5 TASK 2-4: LABORATORY INVESTIGATIONS

Laboratory investigations are performed on samples collected during the research project. Currently, laboratory investigations are underway for the three approaches that the team has collected samples from. All sites are investigated for soil classification (grain-size distribution and Atterberg limits). Further testing is based on possible causes depending on the conditions and design of each site. As of September 2011, particle-size distribution and Atterberg limit investigations have been performed on two borings from SH-6 over Sadler Creek. Results natural water content determinations, Atterberg Limit and grain size distribution testing are presented in Appendix B on page 55 for Sadler Creek.

3.6 TASK 2-5: ASSESSMENT OF FORENSIC DATA

In order to better organize and analyze the collected information, a database has been created. The database contains background, field, and laboratory information collected in Tasks 2-1 through 2-4. Currently, the database is structured in three interconnected levels: “Site Information”, “Bridge Information” and “Embankment Information.”

DATABASE CREATION

Purpose

The purpose of the database is to aid the research team in organizing and analyzing the information gained through the investigation. By putting all the information in an orderly and searchable table, the team can quickly call up facts about the research. The database improves the efficiency of the research. The database was created using Microsoft Access 2010.

Structure

The database comprises of 3 tiers of information. Each tier consists of a table of information and entries in each table are linked to each other; i.e., entries in the “embankment” table are tied to a particular bridge entry, which are tied to a site. As well as these tables, queries can be created that draws specific information from the data tables to answer specific questions or to make comparisons. Forms and reports can also be created to provide a useful interface for data entry, to streamline data collection and make it more uniform, and to present data in a clear and organized fashion. Screen captured images showing various elements incorporated in the database are shown in Figure 23-Figure 28.

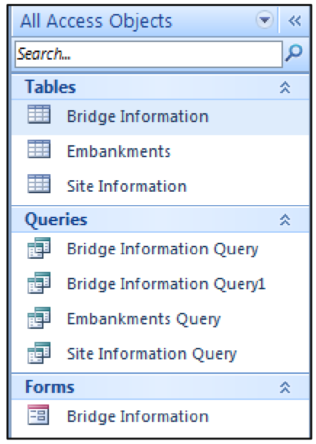

Figure 23 Database Objects

The highest level is "Site Information," which contains basic information regarding the location of the site, who proposed the site, the number of bridges at the site, whether the team has drawings for the site, and when the team first visited each site. An entry is created for each site proposed to the research team.

Site Information | Embankments | Bridge Information

Intersection	Miles from OU	Time from OU	County	ODOT Division	Recommended By	Priority	#Bridges	Over	Visited By	Visit Date	Plans
SH-1 over SH-99	63	1:15	Pontotoc	3	Adam Hill & Eric Cox	1		Road	Colin, Karim, Y	8/26/2011	☐
SH-59A over Big Creek	51	1:00	Pontotoc	3	Adam Hill & Eric Cox	2		Water	Colin, Karim, Y	8/26/2011	☐
SH-1 over BNSF Railroad	73	1:30	Pontotoc	3	Adam Hill & Eric Cox	2		Road	Colin, Karim, Y	8/26/2011	☐
SH-6 North of Retrop	147	2:30	Washita	5	Will Snipes	2		Water	Colin, Karim	6/16/2011	☐
SH-6 over West Elk Creek	133	2:00	Beckham	5	Will Snipes	1		Water	Colin, Karim	6/16/2011	☐
SH-6 over Sadler Creek	143	2:30	Beckham	5	Jerry Clement	1		Water	Colin, Karim	6/16/2011	☐
SH-7 over Beaver Creek	77	1:21	Comanche	7	Original	1		Water	Colin, Karim	6/15/2011	☑
SH-11 over I-35	116	2:00	Kay	4	Original	1		Road	Colin, Karim	6/2/2011	☑
US-177 over Salt Fork River	116	2:00	Noble	4	Original	1		Water	Colin, Karim	6/2/2011	☑
SH-3W over Big Creek	45	1:00	Pontotoc	3	Original	1		Water	Colin, Karim	5/21/2011	☑
SH-48A over Blue River	106	2:00	Johnston	3	Adam Hill & Eric Cox	1		Water	Colin, Karim	5/21/2011	☐
SH-59B over Coon Creek	26	0:50	Pottawatomie	3	Adam Hill & Eric Cox	1		Water	Colin, Karim	5/21/2011	☐
Shields Blvd. over I-35	14	0:25	Cleveland	3	Original	1		Road	Colin, Karim	4/30/2011	☑
SH-33 over Fitzgerald Creek	59	1:00	Logan	4	Original	1		Water	Colin, Karim	4/30/2011	☑
US-62 over Robinson Creek	64	1:12	Lincoln	3	Adam Hill & Eric Cox	1		Water	Colin, Karim	4/30/2011	☐

Figure 24 Portion of Site Information Database

The second level of the database is the "Bridge Information" level. An entry is created for each bridge at each site proposed to the research team. This level contains basic information on the bridge, such as geometric information, traffic information, the date of construction, abutment type, and the severity of the settlement issue. Each bridge entry in this level is tied to an entry in the "Site Information" table.

Site Information | Embankments | Bridge Information

NBI Number	Site	Traffic Direction	Abutment Type	Year of Construction	ADT (2008)
27413	SH-3W over Big Creek	BothBoth Directions	Non-Integral	2005	3800
21980	SH-59B over Coon Creek	BothBoth Directions	Non-Integral	1988	710
26401	SH-6 over Sadler Creek	North Bound	Non-Integral	2000	900
24999	SH-7 over Beaver Creek	East	Non-Integral	1997	3650
24475	US-177 over Salt Fork River	Both, A	Non-Integral	1996	3900
24476	US-177 over Salt Fork River	Both, B	Non-Integral	1996	3900
24477	US-177 over Salt Fork River	Both, C	Non-Integral	1996	3900
27772	SH-11 over I-35	West	Integral	2006	15700
27771	SH-11 over I-35	East	Integral	2007	15700
26280	SH-33 over Fitzgerald Creek	East	Integral	2003	1850
26281	SH-33 over Fitzgerald Creek	West	Integral	2003	1850
21338	SH-48A over Blue River	BothBoth Directions	Integral	1986	1500
27252	SH-6 North of Retrop	North	Integral	2007	340
27253	SH-6 North of Retrop	South	Integral	2008	340
25128	SH-6 over West Elk Creek	Both Directions	Integral	1999	4900
28955	SH-7 over Beaver Creek	West	Integral	2009	3650
28574	SH-99 over Blue River	BothBoth Directions	Integral	2009	980
25511	US-62 over Robinson Creek	BothBoth Directions	Integral	1999	3000

Figure 25 Section of Bridge Information Database

The third level of the database is the "Embankment Information" level. This level contains the most in-depth information. An entry is created for each embankment that the team has visited. Data from field visits, in-situ tests, and laboratory investigations is included in this level as well as pertinent information from background data collection. Data includes information on the bedrock geology, foundation soil, and embankment and backfill materials, measurements from the field visits, and geometric, material, and structural data about approach construction from the design sheets.

Site Information | Embankments | Bridge Information

Embankment	NBI	Slab Rotation	Slab Condition	Slab Min L	Slab Max L	Slab Width	Slab Type
Shields S1	27804		Good	30'	30'	40' 10"	Concrete
Shields S2	27804		Moderate	30'	30'	40' 10"	Concrete
Shields N1	27805		Moderate	30' 7 3/4"	30'	40' 10"	Concrete
Shields N2	27805			30'	30	40' 10"	Concrete
SH-11 E1	27771		Moderate				
SH-11 E2	27771		Moderate				
SH-11 W1	27772						
SH-11 W2	27772						
US-177 A1	24475		Severe				
US-177 A2	24475		Severe				Asphalt
US-177 B1	24476		Severe				Asphalt
US-177 B2	24476		Moderate				Asphalt
US-177 C1	24477						
US-177 C2	24477		Moderate				
SH-7 E1	24999		Good				Concrete
SH-7 E2	24999	none	Good				
SH-7 W1	28955		Moderate	24'	24'		
SH-7 W2	28955		Severe	24'	24'		
6 WEC N	25128	V shape	Severe				
6 WEC S	25128	V Shape	Moderate				Concrete
6 SAD N1	26401		Good				Concrete
6 SAD N2	26401		Moderate				

Figure 26 Portion of Embankment Information

All information is stored in one of these three tables. In addition, there are several queries that can be created to answer specific questions and compare specific aspects. Because the entries in each table are linked to each other, the queries can draw data from each table in a useful manner. For example, a query can make an entry (row) for each embankment, and include information from the correct bridge and site from the corresponding entries in those tables.

Use & Benefits

Information from each task and stage of research is input into the database and used to guide further research. For example, data from preliminary visits and background collection is stored in the database and analyzed to help select sites for in-depth investigations. The organization of the database allows the research team to investigate trends which can help guide the literature review and other investigations.

Site Selection

When selecting sites for in-depth study, a query was created that included all visited bridges and pertinent information such as: ADT, embankment height, issue severity, year of construction, subsurface information, maintenance performed. This information helps to select sites with a variety of issues and construction types.

Sharing Information

The database makes sharing a large amount of information in a more orderly fashion. An easy to use interface has also been constructed to make information easy to find and understand. For example, one page shows embankment construction information with each bridge having a page. Another tab lists all papers gathered for the literature review that can be sorted by author, subject, title, and other parameters; papers with digital copies can be opened directly from the database.

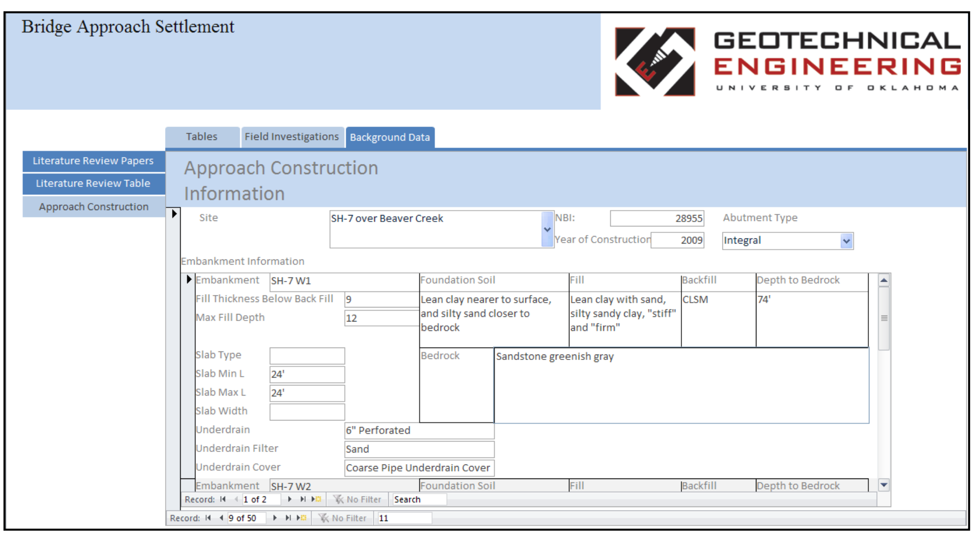

Figure 27 Database User Interface; Embankment Construction Data

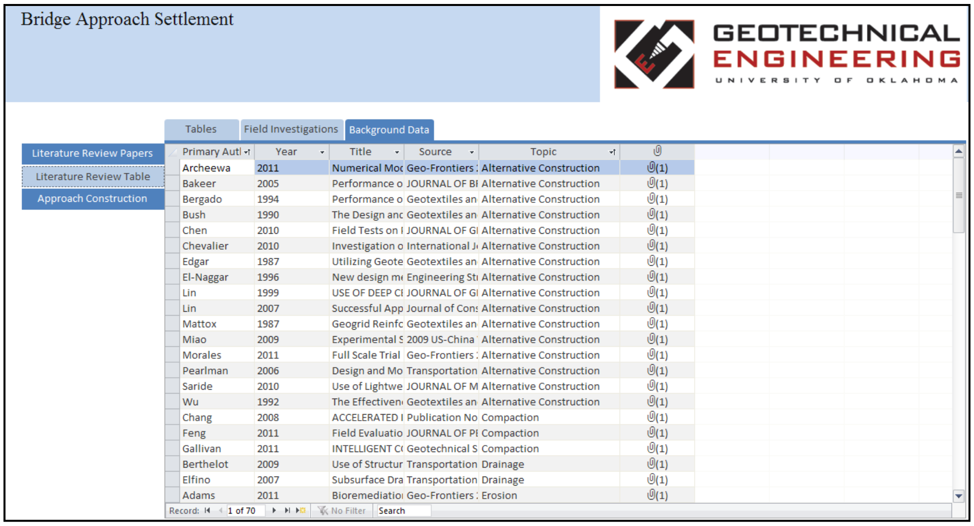

Figure 28 Database User Interface; List of Literature Review Papers

3.7 Task 2-6: Recommendations

There are currently no recommendations. These will be made closer to the end of the investigation.

4.0 Plans for Coming Year

During the coming year in depth study of the 9 sites, and possibly more, will be conducted. The work performed at each site will depend on available information and the additional information desired. It is expected that at least five of these sites will require additional field sampling and in situ testing. Laboratory testing will include standard classification, index property, and compaction testing as well as more advanced testing to study compressibility and wetting-induced volume change of fill and foundation soils. In addition, each site investigated will be analyzed in detail with regard to causes of settlement and bridge/embankment design features to develop recommendations for reducing the approach settlement problem. This will be accomplished in part through further refinement and enhancement of the literature review and evaluation of various alternatives with respect to cost and efficacy.

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Appendices

APPENDIX A: FIELD LOGS

TECUMSEH ROAD OVER I-35

Date	October 15th 2011	Researchers	Karim Saadeddine
Site	Tecumseh Rd. over I-35		Mason Kettler
Boring #	EN-1		

Fill in data for each bag collected

Sample Depth	Water: w.c. tin #	mass (g) wet	dry	tin	Soil Visual Classification	Sample wet-mass (g)	Comments
0-1.5	9	34.74	31.03	11.31	Red/Brown dark clay, moist		Located on North of the east approach
1.5-3	15	52.78	45.00	11.37	"		
3-4	8	41.13	35.49	11.26	" and sticky		
4-5	42	74.64	63.62	11.45	Some black spots. Reddish clay		
5-6	12	39.79	34.21	11.33	Some black with red clay, sticky		
6-7	6	45.00	39.93	11.35	Red clay, dryer		
7-8	11	36.81	33.15	11.35	red dry clay		
8-9	35	36.85	32.97	11.38	"		
9-10	32	66.26	57.04	11.28	"		
10-11	25	34.25	29.87	11.45	red moist clay		
11-12	14	58.33	50.43	11.25	"		
12-13	36	57.02	49.02	11.2	"		
13-14	24	60.91	52.39	11.39	"		

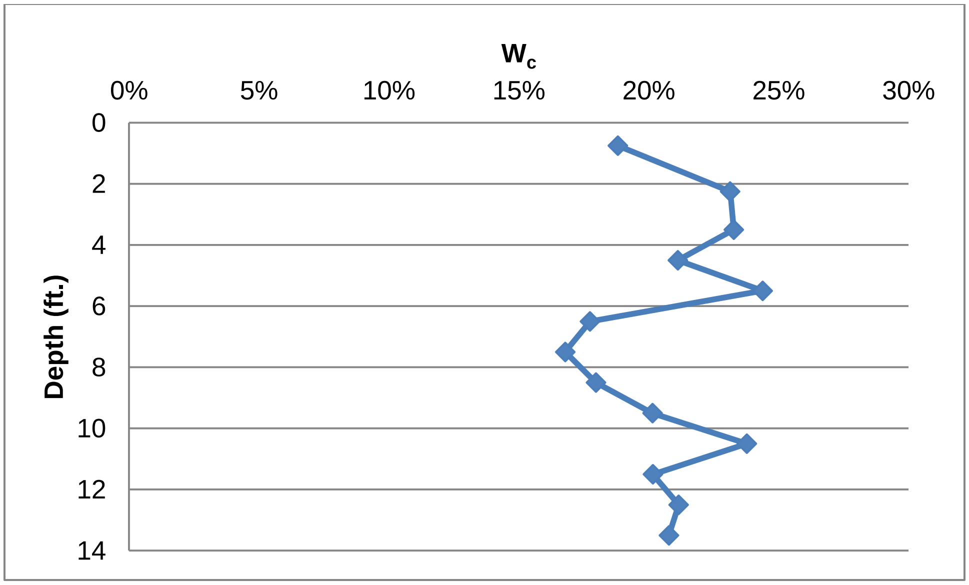

Figure 29 Gravimetric Water Content vs. Depth, Tecumseh Road, Boring EN-1

Date	October 14th 2011	Researchers	Yewei Zheng
Site	Tecumseh Rd. over I-35		Brian Li
Boring #	ES-1		

Fill in data for each bag collected

Sample Depth	Water: w.c. tin #	mass (g) wet	dry	tin	Soil Visual Classification	Sample wet-mass (g)	Comments
1-2	22	44.90	38.94	11.29	Reddish Brown Clay, moist		
2-3	23	32.50	30.04	11.19	Reddish Brown Clay, moist		
3-4	45	60.70	53.16	11.37	reddish/brown silt/clay dry		
4-5	5	45.90	41.87	11.31	reddish/brown silt/clay dry		
5-6	47	45.40	40.65	11.33	reddish/brown silt/clay semi-moist		
6-7	33	38.50	34.14	11.33	Reddish Brown Clay, moist		
7-8.5	13	67.20	58.69	11.37	"		
8.5-9.5	31	46.90	41.35	11.40	"		
9.5-10	2	40.70	36.13	11.28	"		
10-11	37	46.20	41.01	11.29	"		
11-12	26	65.10	57.38	11.42	"		

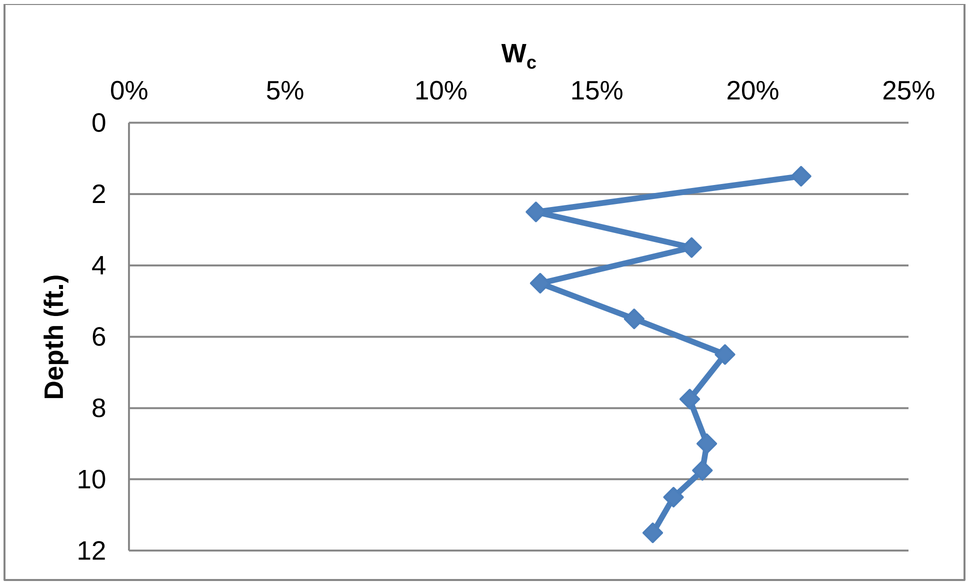

Figure 30 Gravimetric Water Content, Tecumseh Road Boring ES-2

SH-6 over Sadler Creek

Date	July 8th, 2011	Approach	N2	Researchers	Karim Saadeddine
Site	SH-6 over Sadler Creek	Weather	Sunny & Hot		Colin Osborne
Boring #	B1 N1	Auger			

Fill in data for each bag collected

Sample Depth	Water: w.c. tin #	mass (g) wet	dry	tin	Soil Visual Classification	Sample wet-mass (g)	Comments
1-2	P 8	46.42	44.59	16.11			
2-3	Y 6	50.27	46.64	15.98			
3-4	45	46.32	42.95	16.01			
4-5							
5-6	34	37.75	34.47	15.45			
6-7	0 - 6	51.07	45.68	16.12			
7-8	q - 6	41.92	37.95	16.06			
8-9	c - 8	43.09	38.57	15.92			
9-10	t - 9	52.06	46.74	15.91			

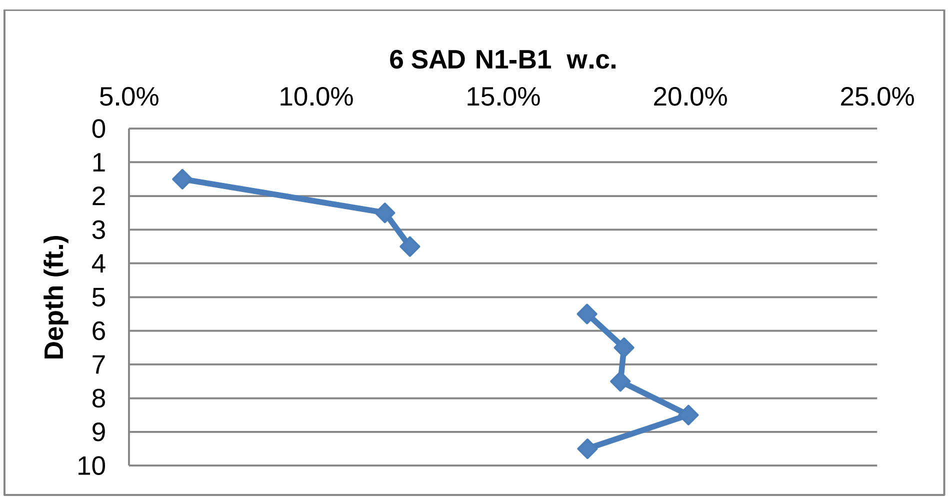

Figure 31Gravimetric Water Content, SH-6 over Sadler Creek, B1 N1

Date	July 8th, 2011	Approach	N2	Researchers	Karim Saadeddine
Site	SH-6 over Sadler Creek	Weather	Sunny & Hot		Colin Osborne
Boring #	B1 N2	Auger			

Fill in data for each bag collected

Sample Depth	Water: w.c. tin #	mass (g) wet	dry	tin	Soil Visual Classification	Sample wet-mass (g)	Comments
0-1	XX10	45.41	42.52	15.94			
1-2	49	64.53	57.32	16.09			
2-3	B-15	48.37	43.15	15.78			
3-4	K-10	49.39	43.35	16.18			
4-5	H10	67.17	59.6	16.16			
5-6	7	39.67	35.96	15.45			
6-7	D-6	61.45	54.32	16.07			
7-8	39	53.9	48.21	15.91			
8-8.5	D-21	46.06	41.71	15.96			
8.5-9	Y-1	58.75	52.13	16.17			
9-10	J-5	50.79	45.45	15.52			

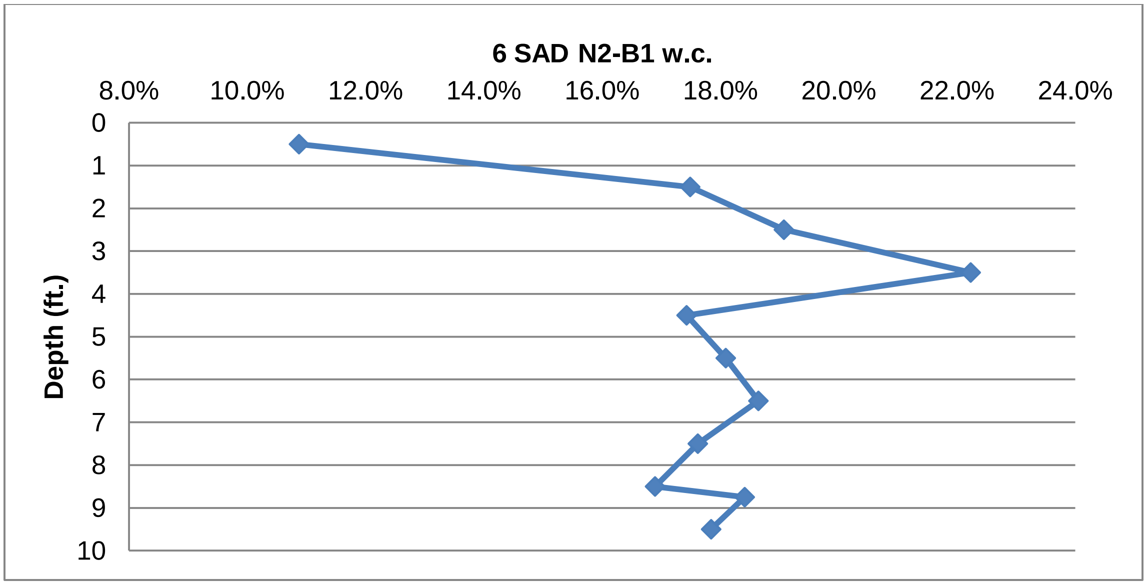

Figure 32 Gravimetric Water Content, SH-6 over Sadler Creek, B1 N2

Appendix B: Laboratory Data

SH-6 over Sadler Creek

Depth: 3-4 Feet

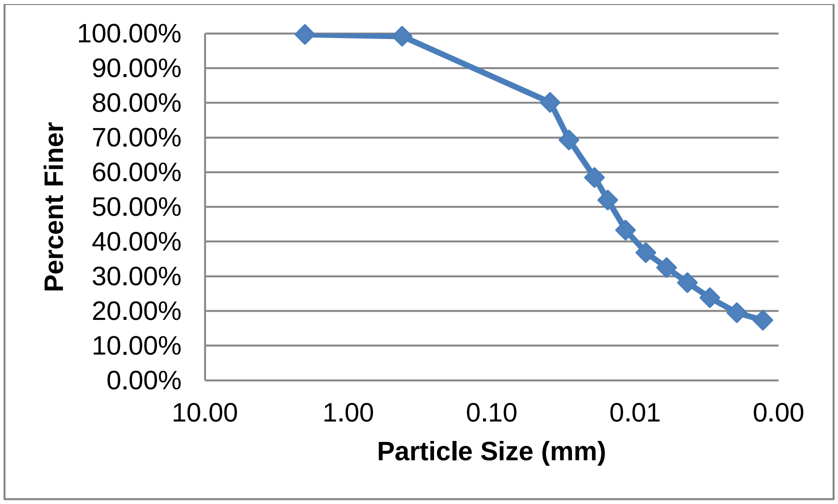

Figure 33 Particle Size Distribution: SH-6 over Sadler Creek B1 N1 3'-4'

Table 3 Atterberg Limit Summary: SH-6 over Sadler Creek B1 N1 3'-4'

Natural Water Content	12.5%
Plastic Limit	18.7%
Liquid Limit	31.1%
Plasticity Index	12.4%

6-7 Feet

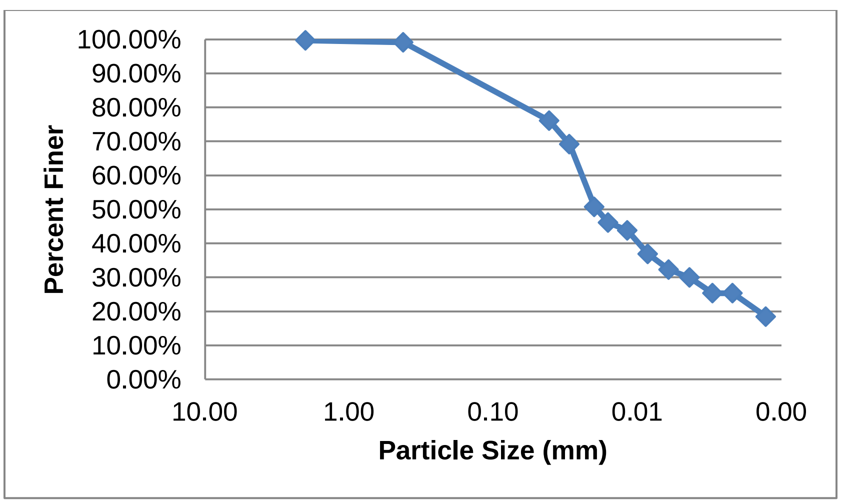

Figure 34 Particle Size Distribution: SH-6 over Sadler Creek, B1 N1, 6'-7'

Table 4 Atterberg Limit Summary: SH-6 over Sadler Creek, B1 N1, 6'-7'

Natural Water Content	18.2%
Plastic Limit	19.4%
Liquid Limit	30.4%
Plasticity Index	11.1%

9-10 Feet

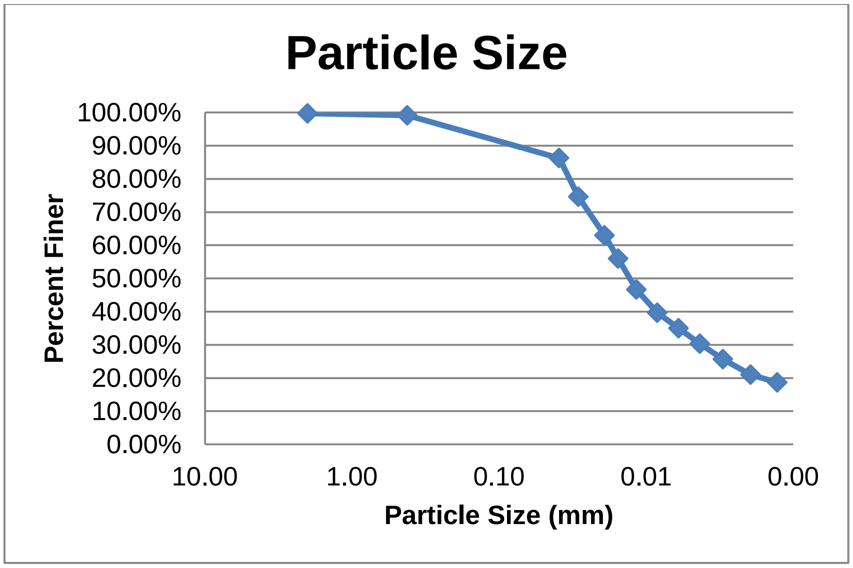

Figure 35 Particle Size Distribution: SH-6 over Sadler Creek, B1 N1, 9'-10'

Table 5 Atterberg Limit Summary: SH-6 over Sadler Creek, B1 N1, 9'-10'

Natural Water Content	17.3%
Plastic Limit	20.2%
Liquid Limit	30.4%
Plasticity Index	10.2%